MODERN STEEL BUILDING CONSTRUCTION.

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I. SOME POINTS IN THE DESIGN. By FRANK N. JACKSON.

IN compiling my notes for this Paper I have endeavoured to describe very briefly the preparation of the designs and drawings for the steel construction of buildings, touching upon some of the rules laid down in Part IV. of the London County Council (General Powers) Act of 1909 which now govern the design of what are called steel-frame buildings. For whilst the provisions of the 1909 Act are in the main excellent and will undoubtedly tend to improve the general standard of steel construction in London, there are some clauses for which one cannot quite account and which seem to bear rather heavily upon the cost of the construction. No discretion appears to be left to the District Surveyor, and one can only appeal on certain points to the County Council.

In commencing the design of the steel construction for a building we have to settle with the architect :

1. The loads to be provided for at the various floor levels.

2. The best arrangement of the stanchions and main and secondary beams, keeping in view the requirements of the building at the various levels, and economy and sound design.

3. Key-plans are then prepared, usually to a scale of $\frac{1}{8}$ inch to 1 foot, showing all girders and stanchions. On each plan are shown the girders on the next floor above, and the stanchions supporting that floor.

4. We then proceed to calculate the loads on the various beams, finding the reactions on the supports and the various forces acting on the beams, writing the loads at each end of the girders on the key-plans. All the beams and stanchions are numbered on the plans and in the calculation books, and so all information respecting any beam or stanchion can be readily turned up.

5. We then find the most economical sections for the various beams, using plain rolled steel joists as far as possible, and where the stresses become too great for plain joists, compounds of joists and plates or riveted plate girders with single or double webs. In doing this we must take into account the maximum permissible bending stress, the maximum shear on web and on rivets, the deflection of the beam, stiffness of web, &c., &c.

6. Having arrived at the sizes of the beams, we work out the loads upon the stanchions, adding in at each floor level the reactions from the beams due to dead and superimposed loads. We thus form a table for each stanchion, showing the total weight at each level, and from this design the section of each length to suit the weight to be carried and resist any eccentricity of loading.

7. Having fixed upon the best section for the stanchion and arrived at the total load at the base, we then design the grillage, which is a beam or series of parallel beams. On the top

of this beam we have the load of the stanchion distributed over a short length in the centre, and under the beam we have the equal upward reaction of the concrete foundation distributed over the whole length of the grillage. The algebraic sum of the bending moments from these two loads is the total bending moment on the beam, which must be designed to resist this bending moment and the shear due to half the load. The grillage beams must also be made sufficient as a pillar to take the total load of the stanchion.

8. The grillage completed to our satisfaction, the size of the concrete foundation for each stanchion is then to be determined from the total load and the nature of the earth at the site. This is a very important matter, and should not be finally settled without careful inspection at the site, and it is most advisable to have borings made to ascertain the nature of the subsoil for some distance below the intended level of the foundations.

9. Complete key-plans and detail drawings are then prepared, from which prints can be made and from which exact quantities can be taken.

10. Tenders are then invited, the contractor basing his price upon the weights in the bill of quantities and upon the labour described in that bill and shown upon the drawings, which should be in sufficient detail to give a good idea of all classes of work to be found in the structure.

When the contract for steelwork is let, orders are placed by the manufacturer with the rolling mills, the material to be made to the test specified.

When the material is rolled and cut to lengths, an inspector attends and takes samples of the bars or plates rolled from each cast or melting. These samples are submitted to mechanical tests for breaking, elongation, and contraction of area. Bending tests also are made after the samples have been heated and slacked out in water. These bending tests are most important. I have known of rods delivered for reinforced work standing excellent tests in breaking, but the material was so unsatisfactory that some of the bars cracked when bent cold to an angle of about 110 degrees. Upon objection being raised the contractors had the bars tested for tensile strength only, showing a high breaking stress, and so managed to get the work passed.

Tests being passed, the material goes on to the yard, and should be carefully straightened, set out and drilled, shaped and riveted as required by the drawings. After inspection, which should be as close as practicable during manufacture, the material is either painted or coated with cement wash and delivered on the site to be there erected.

Erecting Steelwork.

The concrete foundations being ready, the grillages should be set up on steel skids and the stanchions hoisted and bolted to the grillages.

The stanchions should have all the girders put in and bolted up as far as the first joint. The stanchions are then carefully lined up, and the floor girders levelled by wedging under each member of the grillage so that the base of the stanchion is in contact with each grillage beam when the floors are level. If this be done it will be found that the girders will plumb the stanchions. After all is level and in line the grillages may be grouted up solid and riveting may proceed. If this course be not adopted, there is a great waste of time and labour and the result is not good.

One sometimes sees men levelling the grillages separately with a straight-edge and carpenter's level. The staff may be 8 to 10 feet long. In the stanchion itself we have a plumb rule 40 to 50 feet long. Even if the grillage be set as level as possible with the staff the tops of the separate beams are never quite true or straight, and will not give an even bearing for the base of the stanchion unless wedged up under it.

Loads on Floors.

These are now fixed, where the work comes under the 1909 Act, and for the lightest class of floor, for human habitation, we have to provide, in addition to the total dead weight of the structure, for a superload of 70 lbs. or $\frac{5}{8}$ cwt. per square foot of floor area. This means that in a bedroom with a floor area of 200 square feet we provide as superload for furniture and occupants of $6\frac{1}{4}$ tons. Allowing, say, $1\frac{1}{4}$ tons for the furniture, &c., this leaves 5 tons for, say, two people. Where there are several stories of bedrooms in the building the superloads on the stanchions may be discounted within fixed limits, so that where we have nine floors of bedrooms the average superload from each room on the stanchion would be only 5 tons per floor, say $1\frac{1}{4}$ tons for all furniture and $3\frac{3}{4}$ tons for two people in each room, or about $37\frac{1}{2}$ cwt. per person.

There seems to be no danger of the floors and stanchions for domestic buildings calculated on these lines becoming overloaded—in fact, they might be used with perfect safety for light warehouse purposes. I am informed that upright grand pianos, occupying 12 square feet of floor, weigh about 6 cwt. each, so that goods of this class stored with usual clearances would not make up the superload of $\frac{5}{8}$ cwt., probably not more than $\frac{3}{8}$ cwt., per square foot.

Taking a floor in which the steel is stressed to $7\frac{1}{2}$ tons, with dead load $\frac{5}{8}$ cwt. and superload $\frac{5}{8}$ cwt., to reach the average breaking stress on the steel the superload would have to be increased seven times. To stress the steel near the limit of proportionality, the superload would have to be doubled.

The superload fixed for office floors is equivalent to 100 lbs. per square foot dead load. I think there are few offices properly so called where this load is ever attained or even approached, except under the safes. With such a provision, plus the dead load, office floors will in future have to be taken at least at $1\frac{1}{2}$ cwt. per square foot of floor area, and in many cases considerably more, as the floor construction with the partitions will in most cases exceed 70 lbs. per square foot.

I am aware that architects have in the past very generally specified an inclusive load of $1\frac{1}{2}$ cwt. for such floors when inviting tenders, but there has been an enormous difference between the ordinary competitive commercial provision for such loads and what is now required under the 1909 Act. I have generally found in comparing such competitive schemes that the maximum stresses on the steel generally were about 10 tons per square inch or more. In the most remarkable case that has come under my notice stresses of 16, 22, and 24 tons per square inch were found, taking the loads given by the designer at $1\frac{1}{2}$ cwt. per square foot inclusive of the dead weight, and this in a London theatre building. These drawings were accompanied by a letter stating that the stresses on the steelwork did not exceed $7\frac{1}{2}$ tons per square inch, the limit now prescribed.

Bending Stresses in Beams under the 1909 Act.

Where the construction comes under the provisions of this Act the maximum bending stresses in the beams must not exceed the defined limit of $7\frac{1}{2}$ tons per square inch extreme fibre stress.

Of course, some hard and fast line must be laid down, but it seems to me that for solid rolled beams a limiting stress of 8 tons per square inch would have been a more reasonable limit to adopt, taking into consideration the fact that in most buildings one-third to one-half of the load upon the beams is absolutely dead or unchanging, and that the live or variable load changes little, and then very gradually. For the floors of buildings generally, a maximum fibre stress of 8 tons per square inch on solid rolled beams of steel made to the British Standard Specification would ensure an indefinitely long life to the structure, and that is what is aimed at. It

has been the practice of many engineers working on sound lines to adopt this stress on rolled-steel beams of 8 tons per square inch, and in riveted girders of all kinds to limit the maximum fibre stress to $7\frac{1}{2}$ tons per square inch, as is now done under the Act.

May I here point out that an extreme fibre stress of 8 tons per square inch in rolled-steel beams of the largest size means an average stress in the flange of the beam of less than $7\frac{1}{2}$ tons per square inch, and in the smaller beams the average stress in the flanges is generally still less?

In riveted beams there are naturally more opportunities for slight defects in the drilling and riveting, and inequalities of stress are more likely to occur owing to minute differences in the application of the stresses through the rivets, so that a little more margin is desirable in riveted work than in a solid beam rolled in one piece to the standard test. This inequality of stress is to be noticed in the testing of reinforced concrete beams, where under test of a beam with four bars, the beam does not fail as a whole, but the bars break one at a time, partly, no doubt, owing to the bars not being equally straight, and partly from other causes, such as variable adhesion or variable height in the beam.

In considering the desirable or proper limits of stress on mild steel we must keep in mind the following points: whether the maximum stress adopted will during the life of the building cause injury to the steel; whether the stress adopted will cause such deflection or deformation as will injure the structure. In mild rolled steel we have a material which is perfectly elastic up to about 12 tons per square inch and whose yield point is about 18 tons per square inch.

We are dealing with a material in which the beams never collapse suddenly when overloaded, as cast-iron and concrete beams occasionally do. Mild-steel beams well held laterally will carry nearly four times the working stress of 8 tons without breaking, and in their ordinary sizes and spans before reaching the yield point of the material will deflect so seriously that they cannot fail to attract attention and give warning of danger.

Compare this with the singly-reinforced concrete beam, which at or near the yield point of the steel will yield and fail suddenly, so that a beam of this kind, with mild steel stressed under working conditions to $7\frac{1}{2}$ tons per square inch, has a factor of safety over collapse of about $2\frac{1}{2}$. The concrete beam again deflects less than the steel beam before giving way and so gives less warning. I have seen such a beam fail suddenly under test with a deflection of only $1\frac{1}{4}$ inch in 15 feet.

I mention this because the floor slab in buildings constructed of steel beams and stanchions is frequently made of reinforced concrete of one form or another, and as parts of the slab are frequently loaded considerably above the average load on the whole floor, the slab should have a margin of safety fully equal to that in the beams and stanchions.

In the same way in railway and other bridges the floors are designed to carry heavy rolling loads, which affect most severely the bearers and cross-girders immediately under the weight. The stresses in some of these bearers will vary between 1 ton and 6 tons per square inch. The stresses in the main girders will probably not vary more than 2 to 3 tons per square inch.

Deflection of Beams.

Where the span of a beam does not exceed twenty-four times its extreme depth, the beam may be stressed up to $7\frac{1}{2}$ tons per square inch under the 1909 Act.

Loading a 12 inch by 6 inch 54 lb. R. S. J. 24 feet-span with a distributed load of 13 tons, the maximum fibre stress is $7\frac{1}{2}$ tons per square inch. The deflection of the beam is about $\frac{9}{10}$ of an inch, say $\frac{1}{320}$ of the span. But if the span exceeds twenty-four times the depth of the beam, we must under the Act at once reduce the deflection limit to $\frac{1}{400}$ of the span, or 25 per cent.

Take, again, the 12 inch by 6 inch 54 lb. R. S. J. 24-feet span, and loaded this time with a concentrated load of 33 tons at a distance of 14 inches from each support: the shear and bend-

ing stresses are still within the limits defined by the Act, and the deflection is increased to 1·8 inch, or about $\frac{1}{160}$ of the span, or about two and a half times the deflection allowed where the span of the 12-inch beam is 24 feet 1 inch.

This will show that we have a fairly wide range as regards deflection, but it seems a pity that one uniform deflection-limit was not adhered to throughout. In my view, a standard deflection for beams of 1 inch in 40 feet and a maximum fibre stress of 8 tons per square inch would have given far better results.

Web Stresses in Beams.

The limiting shearing stress on mild steel is fixed under the Act at $5\frac{1}{2}$ tons per square inch. Thus a heavy 12 inch by 6 inch R. S. J. may be used up to a certain span to carry a distributed load of 66 tons. The web of a beam is to be stiffened where the depth of the beam exceeds sixty times the thickness of the web plate. Few engineers would desire to exceed this limit. The limiting shear usually adopted on an unstiffened web of ordinary proportions is 4 tons per square inch.

Plate Girder Webs. Bearing. [Fig. 1.]

It is not uncommon to find the webs of plate girders kept $\frac{1}{8}$ inch to $\frac{1}{4}$ inch short of the backs of the main angles. This saves expense in ordering materials and saves labour, and in light girders the point is not important. But in buildings, as a rule, plate girders are used only to carry heavy loads, and here it is most important that the webs should be finished flush with the backs of the main angles, at least under and over heavy concentrated loads, such as stanchions. The vertical stiffeners commonly used in single web and in box plate girders are made as close fitting in the root of the main angles as possible, and there is generally some small riveted connection on the stiffener end, but the dead bearing of the concentrated stanchion weights on the web itself is most valuable and should be insisted on. In first-class yards, used to good railway and bridge work, webs are always finished exact to backs of main angles, the top and bottom edges of the web-plates being planed and finished as carefully as the butt joints.

Bearing of Girders on Brackets.

It is not uncommon in buildings to see girders carrying considerable loads of from 10 to 15 tons, supported on an angle bracket $\frac{1}{2}$ inch thick riveted to the side of a stanchion [figs. 2 and 3]. The end of the girder is commonly $\frac{1}{8}$ inch to $\frac{1}{4}$ inch short of the face of the stanchion, and so does not bear fairly upon the vertical leg of the angle bracket. Such connections of weight-bearing beams are altogether to be avoided, and where it is impossible to make the end of the girder fit close to the stanchion, the angle bracket should be packed out so that the whole of the vertical leg may be available for a bearing [fig. 4].

In addition to the bracket bearing the weight, and to any top connection which may be required, the webs of all girders carrying any considerable load should be stiffened over their bearings by angles riveted to both the web of the girder and the stanchion. Another form of bracket sometimes used consists of an angle bracket with vertical tees or angle stiffeners under, somewhat as sketch [fig. 5]. This form of bracket was perhaps a trace of C.I. stanchion design, or possibly was first intended to put the rivets in the vertical flange of the angle bracket into double shear, and so use them to carry twice the load. The bracket also probably was intended as a secondary consideration to increase the length of the bearing of the R. S. J., but its value in this respect is doubtful. It is very difficult to make all these vertical stiffeners fit so closely to the soffit of the angle as to take the load well. Another point is that if the vertical stiffener does fit well up the eccentricity of the load must be measured from the centre of this long bearing and so is increased, and the tension on the rivets in the brackets becomes considerable.

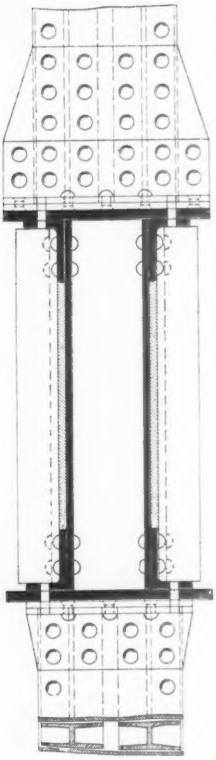


FIG. 1.

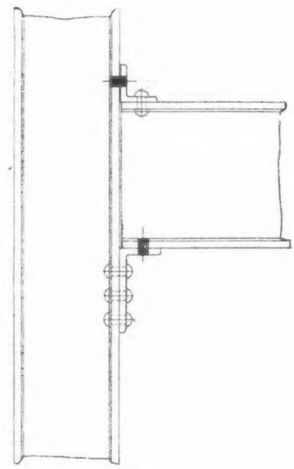


FIG. 2.

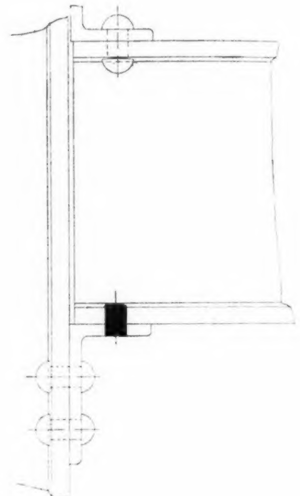


FIG. 3.

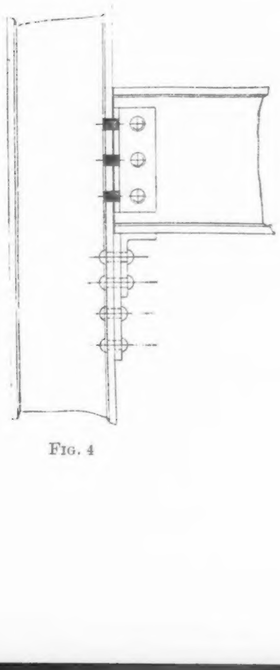


FIG. 4

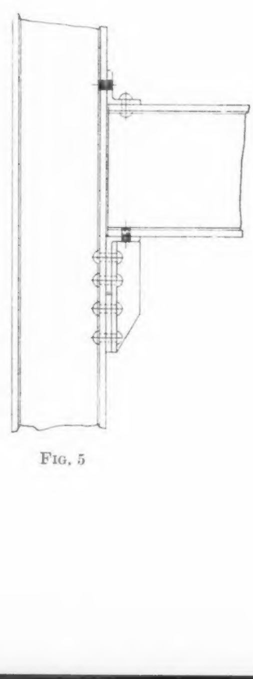


FIG. 5

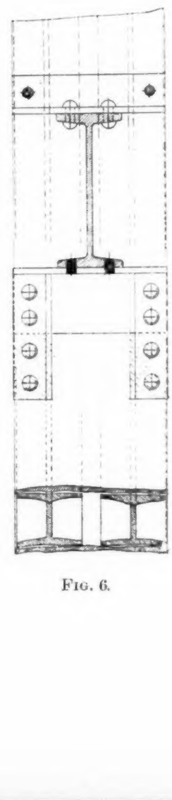


FIG. 6.

Further, these brackets with the vertical tee stiffeners are most unsightly and difficult to case. One sometimes finds the stiffeners under the angle brackets as fig. 6, where they are of little or no use, the angle bracket having to carry as a beam.

Beam Sections.

These again must depend upon the loads and upon their application. Where suitable the single-web section is the cheapest, and the rolled-steel beam is the cheapest of all if it will serve the purpose. We have in the single-web beam the maximum amount of material in the flanges, at the greatest distance from the neutral axis. The next section is the single-web compound girder of R. S. J. and flange plates as required, the riveting in the flange plates being designed to pick up the total stress in the plates.

Where the load becomes too great for a single-web beam of allowable depth, compounds of two or more joists with flange plates will often be found economical. These make a good base for a stanchion or a wall. But where heavy loads are trimmed or connected on one side of such a beam, care must be taken that the half-beam on that side be sufficient to carry the whole of the eccentric load. Where the load is beyond the capacity of rolled-steel beams and their compounds, built up plate girders, with single or double webs, must be used. The general principles are the same, the sizes only being heavier.

Coupling Twin Girders.

This provision seems a little strange. In one part of the Act we are permitted to use beams stressed to $7\frac{1}{2}$ tons per square inch when the laterally unsupported length of the beam does not exceed thirty times the width of its top or compression flange. That is, a 10 inch by 5 inch R. S. J., 12 feet 6 inches long, may be so used. But, should we use two 10 inch by 5 inch R. S. J.'s fairly close together to carry a wall, these two beams, held together by the nature of the load, must be connected at intervals of 50 inches or less with bolts and separators.

A 10 inch by 8 inch R. S. J., 20 feet long, may be used alone, but two 10 inch by 8 inch R. S. J.'s of the same length used together must be connected in no fewer than six places, at intervals of 4 feet 2 inches or less.

The lateral stiffness of the beam is governed by the width of the compression flange and not by the depth of the beam. The lateral moment of resistance of the 10 inch by 8 inch beam is about four and a half times that of the 10 inch by 5 inch beam. The 9 inch by 7 inch rolled-steel beam again is about six and a half times as stiff sideways as the 9 inch by 4 inch beam, but each must be bolted and stiffened at intervals of 3 feet 9 inches or less when used as pairs. Of course, when we have a pair of beams with a concentrated load trimmed in on one side, one should always introduce a connection to equalise the load on the two beams, and where a stanchion or pier is to be carried the beams should be connected, and connection at the bearings of the twin girder are also desirable, but what is stipulated by the Act is, in my opinion, beyond reasonable requirements.

Floor Slabs and Deflections.

Seeing that the provision for live loads on floors is definitely fixed it is incumbent on the designer to keep the dead weight of the construction as low as possible, and as all steelwork must be filled solid with concrete, the small floor beams must be made as shallow as possible.

This can best be done by making all the small filler joists continuous over the main or secondary beams, reducing the deflection and the stress upon the steel.

The deflection of floor slabs has always been a difficulty. As before stated, where the beam does not exceed in span twenty-four times its extreme depth, it may be stressed under the Act to

$7\frac{1}{2}$ tons per square inch, giving a deflection of about 1 inch in 27 feet for uniformly distributed loads.

Where the span of the beam exceeds twenty-four times the depth, the deflection limit rises at once to 1 inch in 33 feet, so that the designer is almost compelled to make the depth of all non-continuous beams not less than one-twenty-fourth of the span.

That is, a 4-inch joist must not exceed 8 feet in span; a 5-inch joist must not exceed 10 feet in span; a 6-inch joist must not exceed 12 feet in span, and so on.

Where the filler joists cannot be made continuous over both supports it may still be possible to fix them at one end and so greatly reduce the deflection and effect economy.

4 by 3 inches R. S. J., 8 feet 6 inches span. Distributed load 1.66 tons. Stress 5.65 tons per square inch. Deflection 1 in 400.

4 by 3 inches R. S. J., 11 feet span. Loaded with $4\frac{1}{2}$ cwt. per lineal foot. Continuous at both ends of span. Total load $49\frac{1}{2}$, say, 50 cwt. Stress $7\frac{1}{2}$ tons per square inch. Deflection about half that allowed.

4 by 3 inches R. S. J. fixed one end only. 9 foot span. Loaded $4\frac{1}{2}$ cwt. per lineal foot $=40\frac{1}{2}$ cwt., say 2 tons. Stress under $7\frac{1}{2}$ tons per square inch. Deflection about half that allowed.

Lapped joists, 4 inch by $1\frac{3}{4}$ inch by 5 lb. [fig. 7]. Say, 11 feet span, $4\frac{1}{2}$ cwt. per lineal foot = $49\frac{1}{2}$ cwt., say $2\frac{1}{2}$ tons. Maximum stress, $7\frac{1}{2}$ tons per square inch. Deflection 1 in 400.

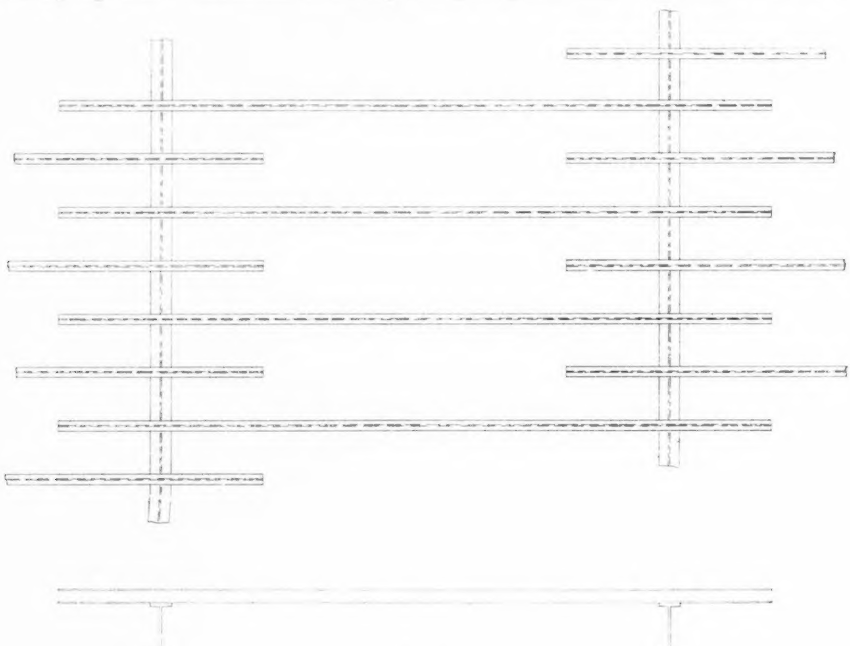


Fig. 7.

This last system may be arranged to form a very economical floor, as at the maximum stress point over support we have the double joist section. At the centre of the span the stress is half that at the supports, and the section also is reduced by one-half. When this system can be adopted it will be found to be very low in cost. This system has the further advantage that

it is not affected by slight inequalities in the heights of the supports when the work is erected as the joists are in short lengths and rest upon two points only.

Where the continuous beams run in one length over three supports, the chances are against these three points being exactly level, but the relatively small depth of the joist will usually allow it to bear at all four points without pressure being applied.

In a non-continuous floor the slabs have a tendency to curve, as fig. 8, and unless the slab is made very stiff the tension in the concrete over the support exceeds its very moderate tensile strength, and cracks appear. To avoid this cracking, wire meshing is sometimes placed in the concrete over the supports. The section of this meshing is not sufficient to fix the ends of the slab or to stop the cracking, but spreads the cracking over a larger area and so hides it or makes it less visible.

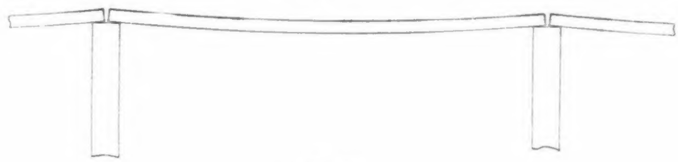


Fig. 8.

Where the floor slab can be made continuous over the supports [fig. 9] the deflection is greatly reduced and the deflection curve is flatter and changes more gradually. The concrete will still crack where the stress on the steel exceeds two tons per square inch, but the cracking will be so distributed along the slab by the steel as to be almost invisible. The ultimate strength of concrete is about 240 lbs. per square inch, say, $2\frac{1}{4}$ cwt. Taking the usual elastic ratio of 15 to 1, where the steel reaches a stress of 34 cwt. per square inch, the concrete will have reached its ultimate strength and will crack.

Fig. 9.

Where the continuous slab is made of reinforced concrete, the bars should be continued over the supports far enough to allow of them developing their maximum working stress over the supports. It is not sufficient merely to hook the reinforcing bars round the flanges of the main beams [fig. 10] as is sometimes done. One cannot be certain that the rods will all bear equally tightly against the beam, and they can only bear on the corners of the flanges. The extension of about 5 diameters with the concrete holding only partly round the bar is too small to be of any real value. To run the bars on, not only gets a better hold, but puts more useful material into the floor.

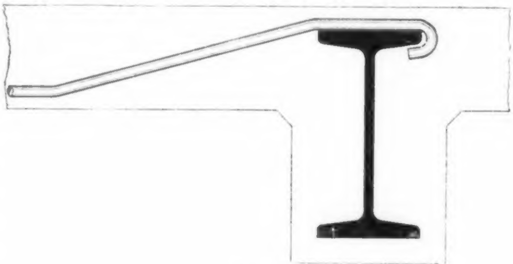


Fig. 10.

In a steel beam of uniform section, uniformly loaded and fixed or continuous at both ends, the maximum bending moment over the supports is two-thirds of the maximum bending moment of a non-continuous beam of equal span similarly loaded, that is, $\frac{WL}{12}$. The stress at the centre of the steel continuous beam is $\frac{WL}{24}$.

In the concrete beam the stresses are not so well defined. When the beam is free at both ends the stresses are only to be determined approximately. It is true that from given data we can calculate the position of the neutral axis to the thousandth part of an inch if desired, but the ramming of the concrete may displace small bars $\frac{1}{4}$ inch or more, and this cannot be seen in the finished work. The strength of the concrete varies within wide limits.

When we come to continuous beams of this material the stresses are still less definite and we are advised by the Joint Reinforced Concrete Committee to assume that the stress to be provided for throughout a reinforced continuous beam should be $\frac{WL}{12}$, and not $\frac{WL}{24}$ in the centre of the span as in the steel beam.

Stanchion Sections.

The sections of stanchions must be made to suit the application of the loads and to transmit them as directly as possible to the foundations.

Thus where we have twin beams resting upon a stanchion, the stanchion itself should,

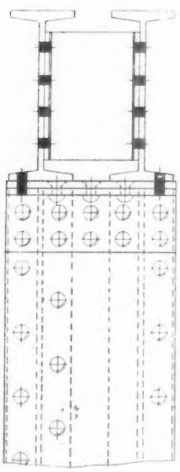


Fig. 11.

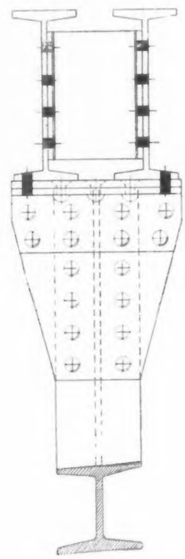


Fig. 12.

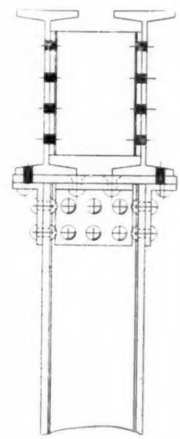


Fig. 13.

where possible, be made of two members whose webs will stand directly under the webs of the beams [fig. 11] and so give ample bearing area for the load.

When a single stanchion is used to carry twin beams [fig. 12], all or nearly all the load has to be carried by the projecting gusset plates, which is not so good, and there is not the same stiffness in the stanchion to resist accidental eccentricity of load.

Twin beams may also be made to bear along the flanges of the stanchion [fig. 13], and here again we get direct bearing of metal on metal and do not depend so much upon rivets in the stanchion gusset plates to take the load.

Taking the various forms in order of cost, the most expensive section is the solid rolled [fig. 14], as the least radius of such a section is small and sound connections are costly.

Next in order come riveted sections of angles [fig. 15], tee bars, &c. [fig. 16], where again the least radius is small and connections expensive. Single web [fig. 17] sections of plain rolled

steel beams and compounds of beams and plates [fig. 18], in the sizes generally used for stanchions, are good and cheap to connect to.

Sections made of three rolled steel [fig. 19] beams are in some cases found very convenient and have a large least radius. Sufficient space should be provided between the flanges of the beams to allow of machine riveting wherever possible.

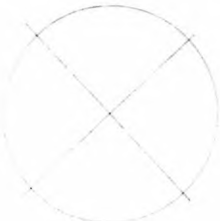


Fig. 14.

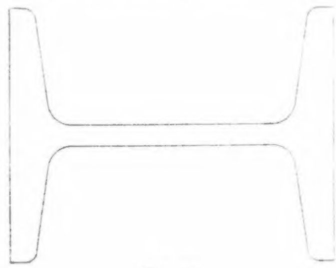


Fig. 17.

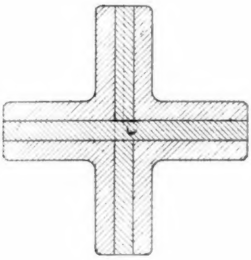


Fig. 15.

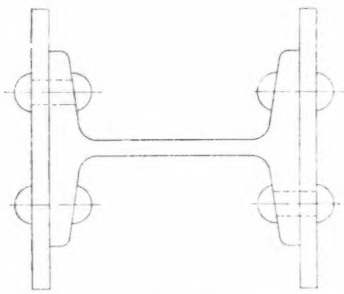


Fig. 18.

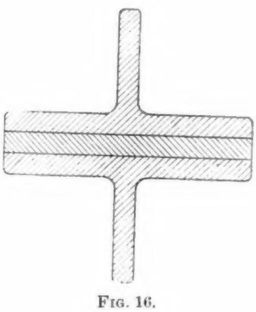


Fig. 16.

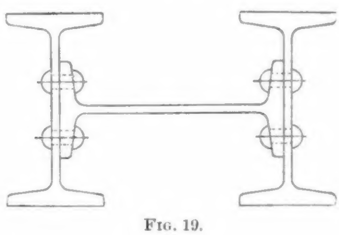


Fig. 19.

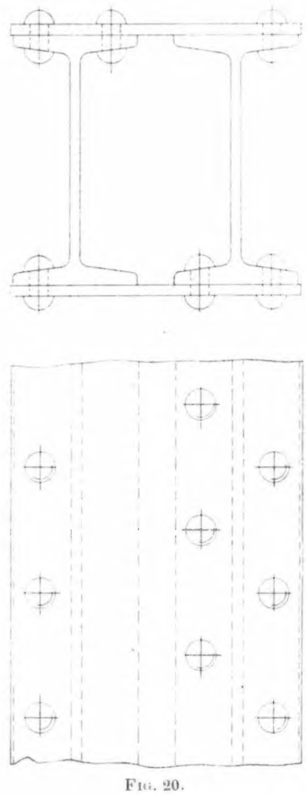


Fig. 20.

The cheapest section generally is made of two R. S. beams with flange plates either in short lengths or continuous riveted on [fig. 20]. Such sections can be arranged with a large least radius, and if the riveting be double in alternate flanges the stanchion is much sounder and stiffer against twisting.

Eccentric Loads on Stanchions.

Few stanchions are quite free-ended, or pin-ended as it is termed, and no stanchion is exactly centrally loaded and so free from bending stresses. If it were possible to make the shaft of a stanchion absolutely homogeneous and perfectly straight and apply the load on the exact centre, a very slender column could be stressed in compression up to the elastic limit of the material.

But the material we have to deal with in stanchions is mild steel of various sections, varying slightly in density and in hardness, and the bars again vary in thickness and in width.

After rolling, the bars are cut into the required lengths and straightened, then set out, drilled, shaped, and riveted. The resulting product is our stanchion, never absolutely straight or uniform in all respects about its geometrical centre.

The formulæ from which the ultimate loads on struts of various kinds are calculated are derived from the mathematical analysis of the results of a number of tests made by various experimenters. These formulæ vary within certain limits, and those laid down under the 1909 Act for stanchions fixed at one end are about a fair average, taking into account the conditions obtaining in the construction of steel buildings, see fig. 20A.

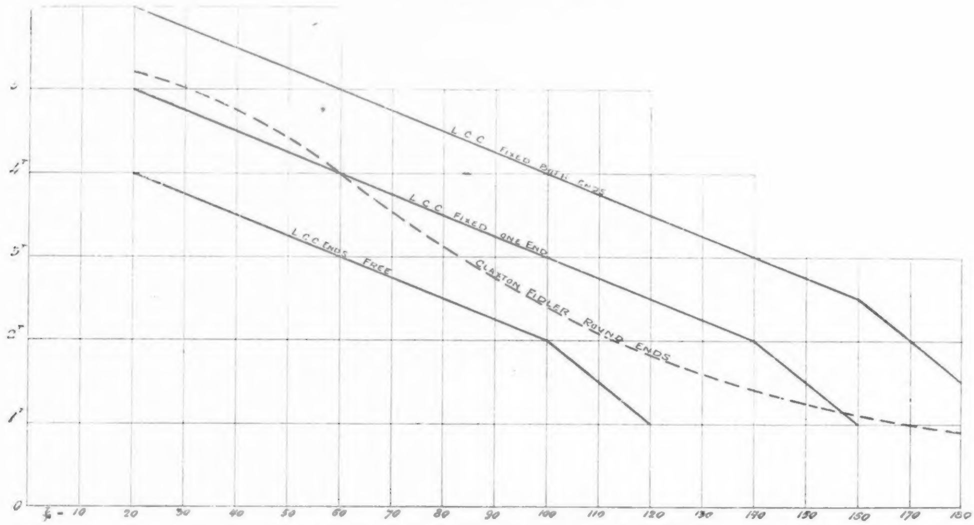


FIG. 20A.—DIAGRAM SHOWING STRESSES ON STANCHIONS.

The limits fixed for stanchions with both ends free are in my opinion altogether too low as there are no absolutely free ends in building work where the general conditions are partial fixing at both ends, equivalent to one end fixed. The bending stresses due to slight inaccuracy in the application of an intended central load on a stanchion, say, one-tenth to one-eighth of an inch out of the straight, are practically negligible. But when we have unbalanced loads supported on a bracket whose centre is 5 inches to 6 inches or more from the centre of the stanchion, as often occurs, the stress in the shaft becomes considerable. Take for instance a 9 inch by 7 inch standard beam carrying a load of 20 tons on a bracket riveted to one flange. The bending moment will be, say, 20 tons by 5 inches=100 inch tons. The modulus of the section being 51 square inches:

The bending stress on the shaft is $\frac{100}{51}=1{\cdot}9$ tons per square inch.

If the permissible stress on the stanchion for the length in question be 3·0 tons per square inch, the maximum stress is

Due to bending	1·9	tons per square inch
,, to load	1·18	,, ,,
	3·08	tons per square inch

The average compressive stress is at the same time only $1\frac{1}{2}$ tons per square inch; but this does not comply with the Act.

If the Act had allowed a little margin for eccentric load, say 1 ton per square inch on the maximum stress, the maximum stress might have been under a load of $26\frac{2}{3}$ tons:

Due to bending, 2·53; due to load, 1·57=4 tons per square inch on the compression side.

On the opposite side of the stanchion the stress would have been

$$\begin{array}{r} +\ 1{\cdot}57 \\ -\ 2{\cdot}53 \\ \hline -\ 0{\cdot}96 \text{ tons} \end{array}$$

The average compressive stress on the shaft would be 2 tons per square inch.

We have here in this stanchion an ⊢⊣ beam of which one flange is stressed in compression to 4 tons per square inch and the other flange bears a tensile stress of nearly 1 ton per square inch. Under the provisions of the 1909 Act the same section might be used without lateral support for a span of 30 by 7 inches=17 feet 6 inches to carry a concentrated load of 27 tons at 14 inches from each support. For a distance of 14 inches from each end the average stress in each flange is $3\frac{3}{4}$ tons. For the central bay of 15 feet 2 inches the maximum stress in each flange, tensile and compressive, is $7\frac{1}{2}$ tons. The length of the compression flange and web is about 110 times the least radius, and the beam is under much the same conditions as if it were acting as a stanchion for a length of 15 feet 2 inches with an eccentric load producing equal tensile and compressive stresses of $7\frac{1}{2}$ tons in either flange. This compares very badly with the limits fixed for the stanchions.

I do not for a moment contend that it is right to use beams stressed up to $7\frac{1}{2}$ tons for a laterally unsupported length of thirty times the width of the compression flange, but I am of opinion that a further margin of stress might very properly have been allowed for the eccentric loading of stanchions.

The case seems particularly hard when in a sound stiff stanchion section, loaded as sketch [fig. 20B], the stress at the extreme corner of one flange is found to exceed by a minute quantity the stress fixed by the Act, whilst all the rest of the sections, probably ninety-nine hundredths of the whole, is within and mostly far below that stress. It is impossible for an area of possibly one-quarter of a square inch stressed to 6 tons to buckle a stanchion of 50 square inches with an average compressive stress of 5 tons.

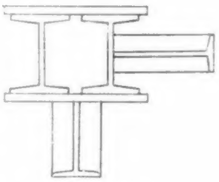


Fig. 20B.

Riveting and Bolting.

Where the conditions admit of sound work, riveting is undoubtedly the best form of connecting the various parts of the structure. But before riveting it is absolutely essential that all the parts should be tightly bolted up so that the various parts are in exact contact before the rivets are driven. If this be not done, of the rivets in a group, the tendency is for those last put in to loosen those first driven.

But where rivets cannot be readily and speedily inserted and headed, good sound bolts are to be preferred. This is especially the case where we have only one hole in a cleat, as it is very difficult and often impossible to get that cleat tight against the plate to which it is to be connected.

Where bolts are used in the finished work the thread of the bolt should be kept clear of the holes in the plates and angles, so that a solid bearing on the shank of the bolt may be obtained [fig. 21].

The thread of the bolt is cut on the solid shank and rounded off at the external angles so

that in the first place the screwed part of the bolt is smaller in diameter than the shank and so does not fill the hole so well. In a bolt one inch diameter the thread or screwed part of the bolt is about one-tenth of an inch smaller in diameter than the solid shank. Again, the bearing area on the edges of the screw thread is extremely small, and this and the small diameter of the thread will result in excessive movement when once the resistance due to friction is overcome.

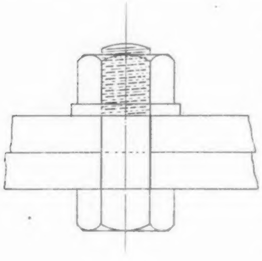


Fig. 21.

The usual length for screwing of bolts is twice the thickness of the nut, so that with stock bolts there is usually from half an inch to an inch of screw thread in the hole in the plates or bars connected. Bolting up requires considerable care, especially when there are several bolts in a group. The bolts in such a case have to be tried over with the spanner again and again to make sure that all are up, as each bolt screwed up brings the plates, &c., into closer contact and the earlier bolts put in are slacked off.

Turned Bolts.

Turned bolts of good quality make excellent work in skilled hands. The holes should be bored to an exact fit and the bolts should be well oiled and twisted in, and backed up by a light hammer. If the bolts be very tight a slight burr or fin is turned up on the shank as the bolt is worked into the hole. So that this burr may not get under the head and keep the bolt from being fully driven up, a small groove is cut round the shank of the bolt under the head so that the fin may turn down into this. The thread of the bolt should, of course, be kept clear of the hole.

Turned bolts are sometimes used to connect steelwork where the noise of riveting is objected to, but I have known at least one case where the turned bolts were so tight a fit that hammers had to be used freely and the noise was quite as bad as hand riveting and more protracted.

Turned bolts should have turned shanks, and not be black bolts skimmed up and ground bright as is sometimes done. These last are generally under size and not round.

Rivets and Bearing Area.

Under the 1909 Act the bearing area of mild steel is fixed at 11 tons per square inch, that is to say, that if the pressure of one piece of mild steel upon another is 11 tons, the area of the surfaces in contact must be not less than one square inch. This is about the average practice.

In riveted work the bearing pressure of the rivet on the plate is naturally governed by this regulation, so that with a rivet 1 inch diameter connecting two $\frac{7}{16}$-inch plates in single shear as fig. 22, we have the shear ·7854 by $5\frac{1}{2}$=4·3 tons. Bearing value, 11 by $\frac{7}{16}$ square inches= 4·8 tons.

But when the rivet is in what is called double shear, as fig. 23, we should have in the ordinary way, shear, 2 by 4·3 tons=8·6 tons; bearing value $\frac{7}{8}$-inch by 11 tons=9·6 tons; but we are not allowed to apply the rule here for bearing area and shear, but an arbitrary limit has been imposed under the Act, fixing the shear on the rivet in these conditions to one and three-quarter times the same rivet in single shear. One cannot see the reason for this.

The single shear connection of the simplest kind under extreme stress tends to take the form in fig. 24, bending the bars connected, and pulling at the rivet heads.

The double shear connection on the other hand will always keep straight under a pull and the riveting is stressed under the best possible conditions.

If we have to design a girder to carry a given load and find that our riveting at one and

three-quarter times the single shear is not sufficient, but that twice the single shear would be ample for our purpose, we may comply with all the requirements of the Act by constructing the web of two $\frac{3}{8}$-inch plates instead of one $\frac{3}{4}$-inch plate, though I cannot say that this would be an improvement in the design [see figs. 25 and 26].

Rivets in Gusset Plates on Stanchions [fig. 27].

The provision for this seems excessive. When the shaft of the stanchion comes directly over the grillage beams, the load from these grillage beams is carried direct through solid metal and rivets seem unnecessary, except to take the reaction from those members of the grillage

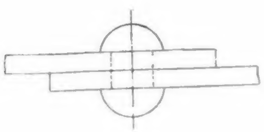


Fig. 22.

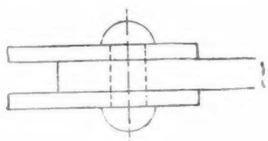


Fig. 23.

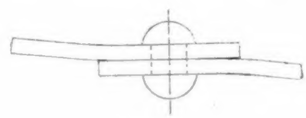


Fig. 24.

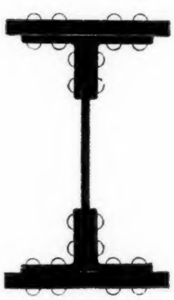


Fig. 25.

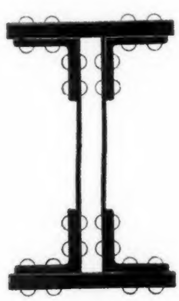


Fig. 26.

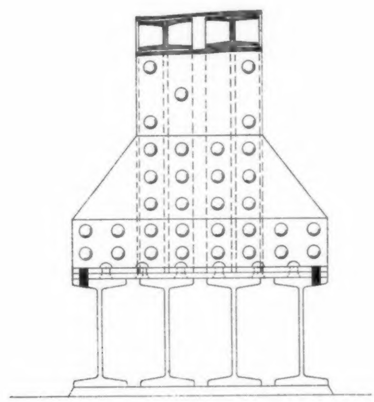


Fig. 27.

which lie entirely outside the stanchion shaft and must be carried by the gusset plates only. This clause may have been taken from American practice, where, I understand, the end of the stanchion shaft is kept clear of the base plate, and all the weight must be carried by the end angles and gussets.

Riveting generally.

There is a limit to the lengths of rivets which can be used, varying with the diameter. The rivet hole is first drilled $\frac{1}{32}$ inch to $\frac{1}{16}$ inch larger than the nominal diameter of the rivet, and the rivets are made from bars rolled $\frac{1}{32}$ to $\frac{1}{16}$ inch smaller than the nominal diameter. The solid head of the rivet is formed in a machine while the bar is hot, and the shank near the head of the rivet is swelled or increased in diameter by the pressure applied in forming the head. So when the hot rivet is first placed in the hole in the plates, it is a good fit near the head and clear of the sides of the hole for the remainder of its length. To form the second and final head and fill the hole in the plates in heavy work a machine is used, power being supplied usually by hydraulic pressure or by compressed air.

Through ordinary thicknesses of metal it is easy in this way to get sound tight rivets which

fit the hole accurately throughout their length when hot and grip the plates tightly when cooled. But when we get a thickness or grip of 5 to 6 inches in a girder flange the case alters. The amount of metal required to form the head and fill the deep hole increases and has to be provided in the end of the shank standing out to be riveted down. The power required is increased considerably, and great care must be exercised or the machine and the shank will go over sideways and form a bad eccentric bend and will not fill the hole. The holes for riveting in heavy work are usually and best drilled through the various thicknesses at one operation by twist drills so that the hole is quite smooth and true throughout. The parts should, of course, be carefully straightened and bolted together through tacking holes before drilling. The smaller the clearance in the hole, the better the riveting will be, but the usual clearance is $\frac{1}{16}$ inch, though the hole is sometimes drilled $\frac{1}{32}$ inch only larger than the nominal diameter of the finished rivet.

In one case where specially exact and rigid work was required, the holes in the plates were bored with a cylinder drill and the rivets turned in a lathe and made to fit the hole as closely as possible when heated. The result was excellent.

Punching the holes is not to be recommended where the material is thick or where many plates have to be riveted together. Punched holes, always more or less taper, the burr being partly cut and partly burst out by the punch. The rivet, even when well heated and closed with a powerful machine, cannot bear well in such work. The effect of the punching in straining the bar may be seen in the curvature of a long angle bar in which a series of holes of close pitch have been punched.

Hand-riveting should not be used where the machine can be employed. The heads are not so sound and the shanks do not fill the hole so well. Hand-riveting is much slower. The machine will perform the whole operation in a few seconds, whilst the hand-riveters take a full minute from first to last, during which the rivet is touching or surrounded by a mass of cold metal. To appreciate this we should see hand-riveting in frosty weather—how the hot edges of the rivet head turn black at once when they touch the cold plates.

The inspection of hand-riveting should be very close; all rivets should be sounded with a small hammer and the heads carefully examined for signs of caulking, fulling, &c.

Bracing of Buildings.

Wind bracing in steel buildings has to be provided to take the place of the cross-walls found in other types of buildings, and should be arranged at intervals of 40 feet to 60 feet, according to the depth of the building, the floors acting as horizontal girders carrying the wind-pressure between the planes of the bracing.

The bracing usually takes somewhat the form shown in fig. 28, a pair of the stanchions being connected by horizontal and diagonal bracing to form a steel tower. On the windward leg we have the live and dead loads on the structure as compressive stresses, less the tensile stresses due to the wind. On the lee leg of the tower we have compressive stresses from both vertical loads and from the wind. The braces are usually made of angle bars or of rolled steel H or ⊔ sections.

Stacks in Buildings.

Owing to the great height of modern buildings, where we have sometimes nine floors of rooms above the ground-floor story, stacks of brick flues built in the old way become very heavy, weighing some 200 tons for each double-flue stack.

As these stacks very generally start at the first-floor level this is a considerable dead weight to be carried over the spans of 40 feet or thereabouts—which are now commonly adopted over

ground-floor public rooms. In such a case, we should often have to carry on a beam 40 feet span two stacks each of about 200 tons and two stanchions each of about 250 tons, say, 900 tons in all, or, with the weight of the beam, possibly 1,000 tons. This would call for a heavy girder of a depth not acceptable to the architect or his client.

Radiators are not popular with the general public in this country, and one finds that in bedrooms especially people like to have an open flue in the room. To get over these difficulties electric radiators have been adopted in some cases with ordinary-looking fireplaces, and the flues have been made of thin slabs of light breeze concrete carried on the floor at each level. This effects an enormous saving in space, weight, and cost. In one large hotel building, thousands of pounds have been spent in building and casing some hundreds of brick fireplaces and their flues, and perhaps of all these bedrooms not more than ten have fires lighted at the same time.

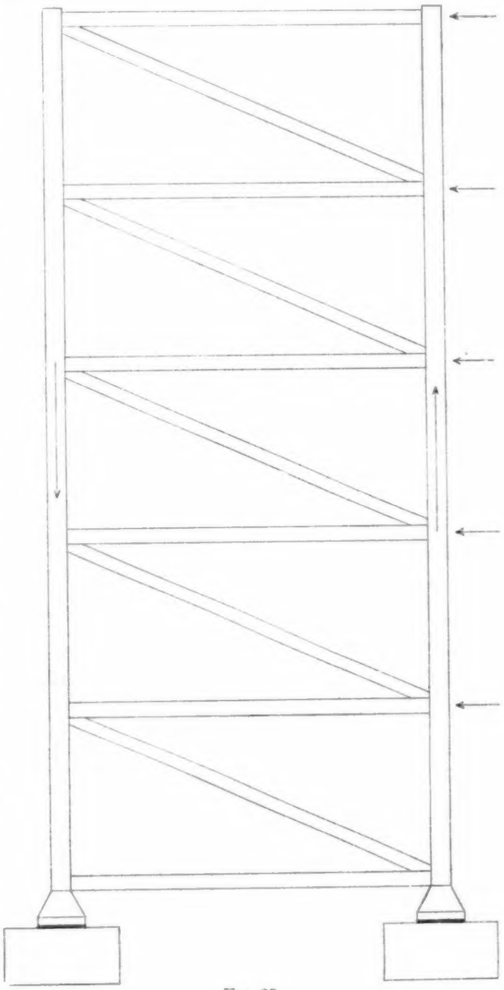


Fig. 28.

Roof Trusses.

These are of many kinds and are made to serve various purposes in addition to supporting the roof and withstanding wind-pressures. They carry trapezes, shafting, and floors, and even form struts to retaining walls.

Roof trusses should be riveted where possible, or put together with tightly fitting black bolts, or with turned bolts where the circumstances warrant the extra cost. Where the stresses vary much or where the roof is open at the sides, all the braces should be made of angle or tee sections, not flat bars.

They should be riveted up at the yard where possible and delivered complete. Large trusses may be made and delivered in sections and riveted upon the site before hoisting.

Roof trusses are usually loaded at the intersections of the braces only, but sometimes the load is distributed along the whole length of the rafter back, which then has to carry as a girder between the braces, being assisted partly by its continuity over those points and partly by the fact that the compressive stress in the back is applied at the gauge line of the bar, which is considerably below the neutral axis of the section as a rule.

Trusses with Curved Ties. (Fig. 29.)

In designing these, care should be taken that the lines of stress in the bottom boom keep well within the section. Theoretically, the curve is formed of a number of straight bars, changing

slope at the intersections of the braces, and large trusses are sometimes made in this way; but it is so much easier and cheaper to bend the bars to a curve that this practice is almost universal. When the truss is loaded the tension in the bottom boom naturally tends to straighten out the curves between the intersections of the braces, and badly designed trusses spread, pushing out

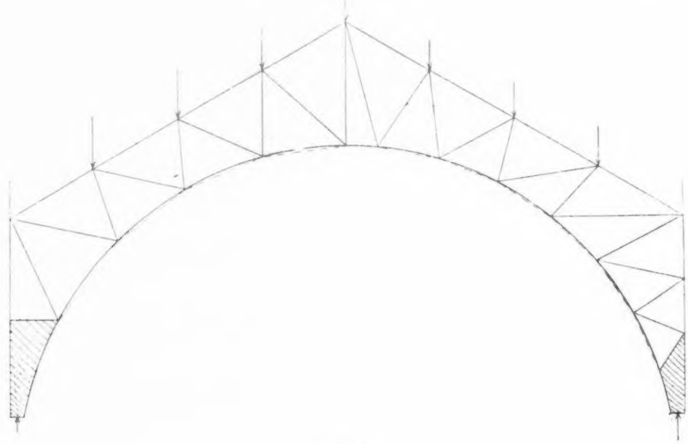


FIG. 29.

the supports, and sometimes causing serious damage. Where this occurs, the roof has to be stripped and additional braces or stiffeners inserted in the truss to help the weak chord.

In the same way the stresses on the angle and tee bars forming struts in the bracing are never applied at the centre of gravity of the cross section, and provision must be made for eccen-

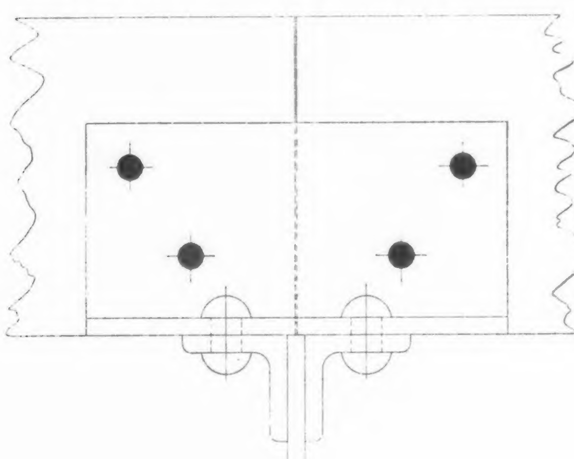


FIG. 30.

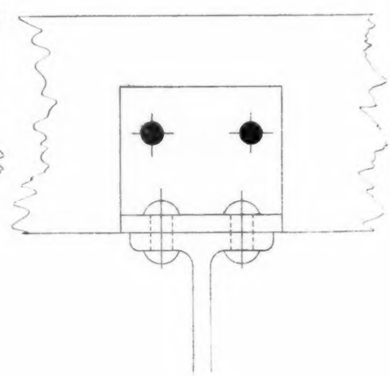


FIG. 31.

tric loading. In struts, sections should be adopted which are equally, or nearly equally, stiff in either direction, and the riveting to the gusset plate may be arranged to stiffen the strut in one direction.

The bracing or stiffening of rafter-backs is, in my opinion, not sufficiently studied. A roof truss is, of course, a trussed beam or girder of great depth, and the width of the compression flange is often not more than 3 to 4 inches for a span of 20 to 30 feet. The section of the compression flange is generally uniform throughout its length, being made sufficient for the highest stress. So we have this very light and leggy truss, often so slight in construction that it has to be stiffened with temporary timber bracing so that it may be hoisted without damage, and the only connection to hold the compression boom straight is a small bolt in a purlin every 7 or 8 feet. It is not surprising that roofs collapse during heavy winds when such construction prevails. The connections to the purlins should be made as rigid as possible, using long cleats and good bolts with large stiff washers (fig. 30). Such a hold is of some real use as bracing, and quite unlike the weak connection, such as fig. 31, often found in such work.

Large roofs should be braced, vertical bracing being inserted between the principals, or the rafters may be braced in the plane of slope. This is especially necessary where the building is open at the sides, and the bracing is of great value in getting the trusses all straight and plumb when erecting.

Bolting Down of Roof Trusses.

This may be done at one or both ends of the truss. One shoe is bolted down through round bolt-holes and tightly fixed. The other shoe has slotted holes which allow of slight movement if expansion takes place, but the bolting is sufficient to allow of considering the truss fixed at both ends to resist wind-pressure. This is done in roofs of moderate spans only.

In many buildings the pier on one side only is insufficient to stand the total wind-pressure on the wall and truss, and so the pier on the lee side has to be brought into action.

Wind Pressure.

Under the 1909 Act an allowance for wind-pressure as a vertical load upon the beams and stanchions must be made of 28 lbs. per superficial foot measured on the slope. The most general slope for roofs in buildings erected under this Act is 75°. With a horizontal wind-pressure of 30 lbs. per square foot the vertical component of this is only 7 lbs.

With 30° slope the vertical component is 17 lbs.
,, 40° ,, ,, ,, ,, 19 lbs.
,, 26° ,, ,, ,, ,, 15 lbs.

So that in most cases we are compelled by the Act to provide for vertical load due to wind about four times as much as can occur.

Pressure on Cement Concrete.

This is now limited to 12 tons per square foot. We find that 6 to 1 cement concrete made from good average ballast will attain a crushing strength of 72 tons per square foot in three weeks, and will continue to increase in strength.

Concrete in steel-framed building constructions is almost solely used as a foundation for the stanchions and piers where it has to bear any considerable stress, distributing the load from the grillage beams on the earth. The direct pressure on the top of the concrete is usually confined to the central portion of the block, the upper surface for a distance of from 1 foot to 3 feet from the edge being unloaded. It follows that the highly compressed part is surrounded and held by a solid mass of concrete, and can only be displaced by direct crushing. I think it would be found on test that concrete loaded in this way and surrounded by earth would resist an enormous pressure. It seems on the face of it very strange that concrete employed under these highly favourable circumstances should be limited to a working stress of one-sixth of the crushing strength at three weeks, while concrete of similar materials in reinforced beams, where the

maximum stress is on the extreme surface of the beam, may be, and often is, stressed up to 30 to 40 tons per square foot. The tendency in such beams is for the concrete to burst upwards when failure occurs.

Blue Brickwork in Cement.

Good blue brickwork in cement has in the soundest engineering practice been loaded up to 16 tons per square foot. The pressure under the Act is now 12 tons per square foot.

Pressure on Earth.

The limits on materials of natural formation, which vary very much in density, are liberal, and not in any way to be compared with the very conservative figures laid down for highly manufactured and uniform materials, like tested mild steel, blue bricks and concrete. Few responsible engineers would care to fix the safe pressures on any foundation without examining it.

As to the pressure of 4 tons per square foot on London Blue Clay allowed under the Act, under test at the Tower Bridge works this gave a settlement of $1\frac{3}{8}$ inches, and this settlement continued as the load was gradually increased to $6\frac{1}{2}$ tons. In the test for Charing Cross Bridge, 3 inch settlement resulted from a load of 6 tons per square foot.

Shoring and Underpinning.

This is a class of work which often comes up in steel construction where alterations are made to existing buildings, and work which requires the greatest care in design and execution.

Temporary timber pillars are usually provided to support the weight where this is of moderate amount, the pillars standing upon timber cills. The cills should be of hard-wood, or hard-wood blocks should be provided under the ends of the pillars to distribute their load over the cills. If this precaution be not taken, heavily loaded timber pillars will sink into the softer side grain of the timber cills.

Needles through walls, under girders, &c., should be of steel and arranged to take a measurable deflection when wedged up. When the calculated deflection is reached and maintained, we may be sure that the load has been taken by the needles, and cutting away may be proceeded with.

Steel planed wedges, well oiled and not too quick in taper, should be used. They are frequently made too short and steep, and are not greased before driving. If great care be not taken with operations of this kind we may easily crack an expensive building from top to bottom, causing damage that can never be made good. I have seen these steel wedges with a taper of as much as 1 inch in 20 inches. One cannot work with such tools.

When inserting new stanchions under existing girders the wedges should be driven up one day and tested again a day or two later if possible, as it is sometimes found that the wedges have slacked out owing to the stanchion foundations taking their bearing. If the wedges be tight and sound, we may be sure that the weight is picked up, and that no settlement is likely to occur. The wedges should then be drilled and bolted and projecting ends cut off.

The work should be examined at each stage, and not left entirely to workmen. I have seen a pier under which it was intended to place a 7 inch diameter solid steel column propped on two 7 inch by 7 inch elm sticks, old stagers, 14 feet long.

Needles are taken out sometimes before the new work under is properly secured. Pillars have sunk into cills and let down and cracked the old walls, and nobody seems to think it anything out of the way.

II. MODERN STEEL CONSTRUCTION: ADMINISTRATION.

By Bernard Dicksee [*F.*].

UNTIL the passing of the Building Act Amendment of 1909 there was practically no legislation or regulations dealing with the iron- and steel-work of buildings. The only provision prior to 1909 was that contained in Section 56 of the Act of 1894 conferring on the District Surveyor power to require, in his discretion, that every breastsummer shall have such additional support as may be sufficient to carry the superstructure.

The provisions of the 1909 Amendment were introduced in order to regulate the construction of "skeleton frame buildings," and are only applicable to buildings of that class of construction. Section 22 provides that "Notwithstanding anything contained in the principal Acts requiring buildings to be enclosed with walls of the thicknesses and of the materials therein respectively described, it shall be lawful to erect, subject to the provisions of this Section, buildings wherein the loads and stresses are transmitted through each storey to the foundations by a skeleton framework of metal, or partly by a skeleton framework of metal and partly by a party-wall or party-walls."

This drafting is somewhat clumsy and confusing, as such buildings have in fact been erected for some years past without special regulation and supervision of the steel-work. Such buildings, however, were required by the Building Act of 1894 to be enclosed by walls of full schedule thickness, although those walls acted solely as enclosures, the loads being carried by the steel frame. Under the 1909 Act these walls may now be of much less thickness, excepting party-walls, which must still be of full schedule thickness.

It will be well to consider the exact application of the 1909 Act, as a difference of opinion has arisen as to what buildings come within the Act. It has been claimed that the Act is an adoptive one; and that, unless advantage be taken of the permission afforded by the Act to build with external walls of less thickness than heretofore, none of the regulations of and restrictions upon the metal framing apply. To accept this view would be to reduce the Act to an absurdity. A builder might produce to the District Surveyor drawings and calculations of a skeleton frame building; upon examination the District Surveyor might find that the loads were in excess of those allowed by the Act; upon the District Surveyor objecting, the builder might reply that, in the circumstances, he would increase the thickness of the walls so as to be of 1894 Act thickness, and would thus get outside the 1909 Act, and build under the 1894 Act only. The effect of this would be to still further load the steel-work, and at the same time to escape the regulations. This question is of such importance that the District Surveyors' Association have thought it advisable to take Counsel's opinion; and they have been advised that such a claim is wrong, but that if, as a fact, the loads and stresses in a building are transmitted through a skeleton framework of metal (with or without party-walls) in the manner described, the whole of the provisions of the 1909 Act apply. In other words, the Act is not adoptive, but compulsory.

It is provided in Section 22 (31) of the Act that:—

"In the case of the erection of a new building of metal skeleton framework, or the making of any addition or alteration or the carrying out of other work under the provisions of this Section, the notice required to be served on the District Surveyor under Sec. 145 of the London Building Act 1894 shall be accompanied (*a*) in the case of a new building, by plans and sections of sufficient detail to show the construction thereof, together with a copy of the calculations of the loads and stresses to be provided for and particulars of the materials to be used, and, should such plans, sections, calculations or particulars be in the opinion of the District Surveyor not in sufficient

detail, the person depositing the same shall furnish the District Surveyor with such further plans, sections, calculations or particulars as he may reasonably require, and (*b*) in the case of an alteration or addition or other work as aforesaid, by such plans, sections, calculations and particulars as the District Surveyor may reasonably require."

As it will manifestly be convenient alike for the architect, engineer, and District Surveyor that the calculations shall be submitted on a uniform basis, thus greatly reducing the labour of making and checking the calculations, the District Surveyors' Association approached the Royal Institute upon the subject ; and, with the co-operation of the Science Standing Committee, they have drawn up a scheme, upon which I am asked to address you this evening.

As skeleton frame construction owes its introduction to the United States of America, inquiries were first made of the New York Building Authorities, but we were informed that no uniform method of depositing plans and calculations was in existence. We are, however, much indebted to the kind assistance of Mr. Robert W. Gibson of New York, who sent us many valuable suggestions, including some of his calculation sheets and drawings. Many of these suggestions find a place in our scheme.

The main points that presented themselves to us were :—

1. A uniform system of nomenclature.
2. A uniform system of symbols.
3. The drawings to be deposited.
4. Calculation-sheets for a uniform method of submitting calculations.
5. Formulæ suggested for use in calculations.
6. A schedule of assumed weights of materials for use in calculations.
7. Tables of safe load for beams and pillars for ready use.

With regard to nomenclature it is proposed that :

Each pillar, column, or pier or point in a wall at which a load is concentrated should be distinguished on the plans, sections, and pillar-sheets by a distinguishing number indicating its position on the plan, and by a distinguishing letter indicating the floor that it supports.

Every point vertically over or under the numbered point on the plan should be known by the same number and by the quotation of the respective storey-letter.

The numbers should be indicated on the plans and be enclosed within a circle to distinguish them from dimensions, and should as far as possible begin at the left hand of the plan and follow across it in continuous numerical series.

The floors should be distinguished by letters as follows :—A should indicate the floor of the ground or entrance storey, or, where there are two storeys that would answer that description, the floor of the lower of such storeys. B should indicate the floor of the storey next above A (*i.e.* the "first floor") ; C the next above B, and so on. X should indicate the floor of the storey next below A ; Y the next below X ; Z the next below Y. Thus Pillar 21 E will be situated in the third storey supporting the fourth floor, vertically over Pillar 21 D supporting the third floor, also vertically over Pillar 21 A supporting the ground floor.

Every girder or beam should be distinguished by the quotation of the numbers at its two ends or points of support, and by the distinguishing letter of the floor in which it is situated. The reactions at the ends of the girder or beam should be distinguished by the same numbers. Thus beam 7-8 B will indicate the beam in the floor of the first storey, one end of which is supported on Pillar 7 B and the other on Pillar 8 B ; the reactions being denoted by R_7 and R_8 respectively, and carried to Pillars 7 B and 8 B.

This method of denoting the beams and pillars will reduce the chance of error to a minimum ; it is almost impossible to carry a load to a wrong pillar.

The Concrete Institute having previously adopted a list of uniform symbols, which are

admirably adapted for general use, these were adopted, a few additions being made to meet the case of terms not required for reinforced concrete.

The method adopted is to take the initial letter of the term for its symbol; this can be done with very little duplication. Small letters are adopted in the case of intensity of loads and stresses; capitals in the case of total loads and stresses. Small letters are employed to indicate inches, capitals to indicate feet.

Section Modulus had to be added to the Concrete Institute list; for this (S having been appropriated for Shear) Z was adopted, and is, I think, already in general use.

Before passing from this subject, may I enter a protest against the misleading, not to say erroneous, use of the term "Moment of Resistance" when "Section Modulus" is meant? Section Modulus $\left(\frac{I}{y}\right)$ is a mathematical property of the figure of the section only, without any relation to the strength of the material, and is not a "Moment" (which must comprise both force and leverage); whereas the "Moment of Resistance" is the product of the Section Modulus by the extreme fibre stress; and may be equated to the "Bending Moment."

It is requested that the drawings deposited should be either photo reproductions, with black lines on white paper (ordinary blue prints are objectionable), or tracings on linen.

A complete set of ⅛-inch scale working drawings showing all the steel construction with properly figured dimensions is asked for, with all the pillars and beams clearly numbered and lettered according to the aforementioned scheme of nomenclature.

But, if desired, the steel-work may be shown upon a separate set of drawings to accompany the ordinary working drawings. Details of the steel-work will also be required.

We now come to the important question of the calculations. The calculations for pillars, for beams and girders, and for foundations should be tabulated upon separate sheets or schedules, on the forms published for that purpose. These forms are of a convenient size—*i.e.* foolscap—and are upon thin paper, which has the advantage of limiting the bulk of a set of calculations, and also of enabling sketches to be inserted by tracing.

Pillar-sheet.—The calculations for each pillar, for the full number of storeys through which that pillar passes, should be tabulated upon a separate pillar-sheet or set of sheets.

The sheet should be headed with the identifying number of the pillar, and the material to be employed, on the left hand; the name or identification of the building in the middle; and the identifying numbers of exactly similar pillars, on the right hand.

The sheet is divided into three panels extending horizontally across the page, each of which panels is to contain the calculations for the portion of the pillar within the limits of one storey; the calculations of each storey being placed in the panel that represents the actual position in the building of that storey, the topmost storey being at the head of the page, and the lowermost at the foot. Where the page does not contain a sufficient number of panels for all the storeys of the building (and that will be the case where the building extends to more than three storeys) the calculations are to be carried forward and continued on to an additional page in the same order, all the pages of one pillar-sheet being carefully attached to each other.

The pillar-sheet is divided into vertical columns, as follows. The first eight columns contain details of loads, &c., and the remaining three columns the results arrived at:—

Col. 1.—To contain the distinguishing letter of the storey, and the height of that storey.

Col. 2.—The particulars of loading set out in detail, collecting the reactions from the various beams, keeping dead and superimposed loads distinct so as to provide for the allowed discount from the superimposed loads.

Cols. 3 and 4.—The amount of the loads described in Col. 2, axial and eccentric loads being kept distinct in Cols. 3 and 4 respectively.

Col. 5.—The totals of the loads from Cols. 3 and 4, showing the total loads for that storey only.

Col. 6.—The total loads from Col. 5 collected storey by storey, showing the total loads (dead and superimposed) down to the level of the floor of the storey.

Cols. 7 and 8.—The arm of eccentricity of loading in two directions normal to each other, having reference to the eccentric loads set out in Col. 4.

Col. 9.—Sketch, approximately to scale (upon the axes shown), of the pillar section decided upon, with dimensions and thicknesses.

Col. 10.—The properties necessary for calculations of the section decided upon, only those actually required being inserted.

Col. 11.—Calculations of the maximum stress due to axial and eccentric loading (where necessary, in two directions normal to each other), and a statement of the maximum permissible load according to the table set out in Section 22 (21).

In making these calculations the pillars should be taken as "one end fixed and one end hinged." Only in cases where special means have been adopted to secure fixing may the pillar be taken as with "both ends fixed."

In the pillar-sheet no provision has been made for any additional working stress upon a pillar at the leeward side of a building due to wind-pressure, as by Section 22 (21) no extra allowance need be made for this extra stress where it does not exceed 25 per cent. of the permissible stress according to the table. An excess equal to this will rarely be produced upon buildings in London; it is therefore thought best not to complicate unnecessarily the pillar-sheet. Where this extra stress becomes a factor to be dealt with, an additional calculation may be made on an additional attached sheet, and the result entered upon the sheet in Col. 2, and carried forward in the totals.

All loads on the pillars should be expressed in tons and decimal of a ton.

Beam-sheet.—The calculations for each series of girders or beams situated vertically over each other and with their ends supported on the same two pillars, and therefore bearing the same distinctive end-numbers, should be tabulated upon a beam-sheet in a similar manner as the respective pillar-sheets. Each series of beams with different end-numbers should be upon a separate beam-sheet.

The sheet should be headed with the identifying number of the beam, the material to be employed and the width of span on the left hand; the name or identification of the building in the middle, and the identifying numbers of exactly similar beams on the right hand.

The sheet is divided into panels extending horizontally across the page, each of which panels is to contain the calculations for one beam or girder in the series. The storey in which the beam or girder is situated should be indicated on the sheet similarly to the corresponding storey on the pillar-sheet.

When the page does not contain a sufficient number of panels for all the storeys, the calculations are to be continued on to an additional page in the same order, all the pages of one beam-sheet being carefully attached to each other and numbered consecutively.

The beam-sheets are in two forms :—(*a*) To be used in cases where by reason of the application of concentrated loads or for other reasons a special calculation is required; (*b*) To be used only in cases where the load is uniformly distributed, and the beam may be selected by reference to the annexed tables of calculated safe loads for standard sections.

Beam-sheet (*a*) is divided into vertical columns as follows :—

Col. 1.—To contain the distinguishing letter for the storey, and the height of the storey.

Col. 2.—The particulars of the loading set out in detail, giving dimensions, weights, &c., for each item of the loading, whether walls, floors, or other beams.

Cols. 3 and 4.—The amount of the loads referred to in Col. 2 (uniformly distributed and

concentrated loads being kept distinct in Cols. 3 and 4 respectively), with the totals at the foot of the panel.

Col. 5.—A letter distinguishing the point of concentration of the load.

Col. 6.—(*a*) A diagram of the beam showing: Uniformly distributed loads (dead and superimposed loads being kept distinct): the points of application and the amounts of concentrated loads (distinguished by the corresponding letter in Col. 5 and by figured dimensions); the span of the beam, the points of reaction distinguished by their proper numbers according to the nomenclature: where the circumstances of the loading render it desirable, a diagram of the moments, set up to scale on the floor-line.

(*b*) A sketch of the section of the beam employed, with dimensions and thicknesses.

Cols. 7, 8, 9.—The reactions, showing in Col. 7 whether the load is distributed or concentrated, and if the latter the distinguishing letter identifying the loads as in Cols. 5 and 6; in Cols. 8 and 9 the consequent reactions of these loads at each end of the beam, distinguished by its proper number. The reactions due to dead and to superimposed loads are to be kept distinct in order to facilitate transference to the pillar-sheets (where an allowance may have to be made in the case of superimposed loads). These reactions should be summed up separately, and the totals of both inserted at the foot.

In the case of buildings of the warehouse class, where no reduction of the superimposed loads is allowed, the reactions due to dead and superimposed loads need not be kept distinct.

Col. 10.—The Maximum Bending Moments resulting from the various loads indicated in Col. 7, and the total Bending Moment in the beam.

Col. 11.—The properties necessary for the calculation of the section decided upon.

Col. 12.—The maximum stresses in the beam resulting from the loading, which stresses must not exceed those allowed in Section 22 (22).

It may here be remarked that the Maximum Bending Moment due to the whole system of loading will not necessarily be the total of the various Maximum Bending Moments due to the individual loads. In fact, it can only be so in the case of a load concentrated at the centre of the beam; in all other cases it will be something less than the total of the various Bending Moments, due to the fact that each Maximum Bending Moment occurs at a different point in the span.

The labour of calculation for beams variously loaded, when Beam-sheet (*a*) is employed, may frequently be simplified and the calculations made without recourse to the properties in Col. 11, by ascertaining the Equivalent Distributed Load that would produce the same Maximum Bending Moment as that produced by the several combined loads, concentrated and distributed. This can be done by an inversion of the formula

$$B = \frac{WL}{8};$$

i.e. Equivalent Distributed Load $= \dfrac{8\,B}{L}.$

Having ascertained the Equivalent Distributed Load, a reference to the Tables of Safe Uniformly Distributed Loads attached to the scheme will at once show the section necessary to carry safely the load over the given span.

Beam-sheet (*b*) is divided into vertical columns as follows:—

Col. 1.—To contain the distinguishing letter for the storey and the height of the storey.

Col. 2.—The particulars of the loading set out in detail.

Col. 3.—The amount of the loads referred to in Col. 2, with the totals at the foot of the panel.

Cols. 4 and 5.—The reaction at each end of the beam, distinguished by its proper number;

the reactions due to dead and to superimposed loads being kept distinct, and the total loads inserted at the foot of the panel.

Col. 6.—The dimensions and weight per foot run of the section selected.

Col. 7.—The safe load according to the annexed tables for the section selected and the stated span.

The loads on beams and reactions should be expressed in tons and decimals of a ton.

The scheme also contains a Foundation-Sheet, which needs only a passing reference.

Roofs should be dealt with by means of reciprocal diagrams of the stresses in the trusses, shown to a convenient scale, accompanied by tables of results.

Calculations for all members for which special calculation-sheets are not mentioned should be shown upon additional sheets, in such detail as circumstances may require. This will include lopsided loading of beams and girders, gusset plates and brackets, bracing, and connections generally.

A number of Formulæ that it is suggested should be employed in making the calculations are included. Some of these are elementary, but it was thought desirable that these should be comprehensive, so that all should work on a uniform basis. The formula for eccentrically loaded pillars will perhaps be the most valuable. As all buildings should be sufficiently braced to ensure that there shall be no material deflection in the pillars, this formula has been adopted in preference to the more complicated one necessary in cases where material deflection occurs.

Not the least useful part of the Scheme will be the carefully worked-out tables of safe loads, in accordance with the limits of stress allowed under the Act, for beams and pillars. The extreme fibre stress allowed for steel ($7\frac{1}{2}$ tons per square inch) considerably simplifies the labour of calculation. It will be observed on reference to Tables 1 and 2 that the coefficient WL works out at exactly five times the Section Modulus. Thus, having ascertained the Section Modulus, it is only necessary to multiply by 5 and divide by the span in feet to ascertain the safe distributed load in tons.

Tables 1 and 2 (Beams) have been worked out for simple standard sections of joists and channels; should compound sections be necessary, reference may be made to similar tables for compound girders in the handbooks published by Messrs. Dorman & Long, and the Cargo Fleet Iron Co., in both of which books the calculations have been made on the same basis of $7\frac{1}{2}$ tons per square inch.

Table 4 (Stanchions and Pillars) has been worked out upon the middle column of the table of working stresses contained in Section 22 (21) *a*; there described as one end hinged and one end fixed. This is the middle position between free ends and fixed ends, and it was agreed between the District Surveyors' Association and the Science Committee that this was the table applicable to ordinary practice. Up to $\frac{l}{r} = 140$ it may be expressed in tons as, Safe load $= 5\frac{1}{2} - \frac{1}{40} \cdot \frac{l}{r}$; or in pounds, Safe load $= 12{,}320 - 56\frac{l}{r}$. It approximates to, but is about half a ton safer than, the American formula of $13{,}500 - 57\frac{l}{r}$.

This table has been calculated for such of the standard sections as are at all suitable for use as stanchions, and does not attempt to deal with compound sections. The handbooks before referred to contain extensive calculations for compound stanchions, but unfortunately they are not available for the purposes of calculations under the Act of 1909, as they are calculated on a different basis from that required by the Act.

Table 3 gives the values of $\frac{l}{r}$ for various heights and radii of gyration.

Table 5 gives the maximum shearing and bearing values in single and double shear for rivets and bolts of various diameters and various thicknesses of plates.

In conclusion, may I express the hope that the scheme that it is my privilege to bring before you to-day may prove to be of considerable use to the members of the Institute? May I also express the hope that it will be universally adopted for all buildings to be erected under the Act of 1909, as such adoption will help to lighten the enormous amount of labour that, for a very inadequate remuneration, has been thrown by that Act on the District Surveyor?

[*Discussion adjourned to Monday 28th April.*]

VOTE OF THANKS.

The President, Mr. Reginald Blomfield, A.R.A., in the Chair.

Mr. ALAN E. MUNBY, M.A. Cantab. [*A.*], Chairman of the Science Standing Committee, in proposing a vote of thanks, said that the suggestion of the subjects of these valuable Papers emanated from the Science Committee, the matter having been discussed by them more than a year ago, which enhanced his pleasure in rising to express the thanks of the meeting to the Authors. They all knew Mr. Jackson and Mr. Dicksee. Mr. Jackson, as an Honorary Associate, possessed one of the highest honours which the Institute could give. He had served the Institute in various ways on the Science Committee: and he, as the Chairman, could personally say that he had served it very well indeed, and had given them most valuable advice on many technical points. They hoped to see him again on the Committee. They knew him also as a designer who could even start with the roof of a building, and finish at the ground. Mr. Dicksee had also served on the Science Committee for some time, and those who knew the District Surveyors' Association could appreciate what he had done for that body. He could imagine that these Papers had left various effects upon different people. He noticed a friend near him who had a look of utter desolation, others who could appreciate the calculations probably had a feeling of elation, and there was at least one Fellow with a feeling of aggression, which he would probably shortly put into operation. He himself was not nearly so competent to deal with the points in the Papers as many other speakers, and it would not be fair for him to take up their time, as the hour was so unusually advanced and the discussion promised to be most interesting. He would therefore content himself by proposing a very hearty vote of thanks to Mr. Jackson and Mr. Dicksee for the great trouble they had taken, and for the excellent and useful information they had put before the Meeting.

Mr. W. G. PERKINS seconded, it being understood that he would reserve his remarks on the subject of the Papers for the adjourned Meeting to be held on the 28th.

The PRESIDENT said they had had a tremendous doing that evening, and it was getting late. As he listened to the relentless and implacable science of Mr. Jackson and Mr. Dicksee he did not know where he was; and he began to think that possibly, as a brilliant Professor recently said, all their studies in Gothic and Classic and other things had been as waste paper, and they ought to have devoted those years of laborious effort to the study of steel construction. He should not, however, throw up the sponge so readily. Nor would he detain them with any remarks that evening, because he felt that there was a tremendous lot which had been said in the Papers that required thorough discussion by competent men. Therefore it had been suggested that it would be desirable to adjourn this meeting and discussion, for the subject was of vital importance, and affected the future of architecture very closely. In putting the vote of thanks he need hardly say that Mr. Jackson's and Mr. Dicksee's Papers had been up to the highest standard of papers which they had learned to expect in the Institute; and when they were published, they should look to them with the greatest interest, and to the discussion which followed, as authoritative on this subject.

The vote of thanks was carried by acclamation and briefly responded to, and the discussion was adjourned to Monday, 28th April.

THE TEMPLE OF EZEKIEL.

By G. S. Aitken, Architect (Edinburgh).

THE following illustrated interpretation of the Prophet Ezekiel's Visionary Temple is the result of a professional study of Syrian architecture undertaken in connection with a course of lectures given in the Heriot-Watt College, Edinburgh, some years ago.

The author's theory is new, and quite opposed to those of past commentators, who, unable to regard the subject from the technical standpoint of the architect, could not reasonably be expected to resolve its difficulties.

Of the various exegetical authorities referred to in the course of his analysis, the author begins with that under the article "Temple" in Dr. Hastings' *Biblical Dictionary*, referring afterwards to the corresponding article in the *Encyclopædia Biblica*, and, as occasion requires, to the views of different authors who have commented on the vision. For convenience of reference the two works will be alluded to as *H. B. D.* and *E. B.* respectively.

The inspired account of the Visionary Temple is given in chapters xl., xli., xlii., xliii., and xlvi. of Ezekiel's prophecy.

In chapter xl. the prophet is represented as being led by his guide to the east side of the Temple enclosing wall. The first measurement taken is of the outer walls, which are found to be 6 cubits in height and breadth.

The outer threshold of the east gate is 6 cubits long; beyond this are guard-rooms or lodges, three on each side, 6 cubits square, divided from one another by massive walls, 5 cubits thick, with a screen or "boundary fence" in front, 1 cubit thick, of the nature of a parapet, with an entrance in the middle; beyond them is a second threshold, 6 cubits long, and outside this a porch 10 cubits wide and 13 cubits long externally, its threshold 6 cubits long with ingoings of 2 cubits (chapter xl. verses 8, 9, 11).

The 5-cubit dimensions between the guard-rooms seem excessive, but, although we are not to suppose the plan was copied from any Assyrian work, we know that the intermediate walls of one of the Khorsabad gates were similarly massive. They were reduced in thickness by the windows, which the sixteenth verse of the same chapter tells us were placed in the "posts" of the chambers; these windows would probably have recesses with seats for the use of the guards, like those we find in the mediæval castles of our own country.

The expression "door against door," accompanying the statement of width in verse 13, implies that the 25-cubit dimension was limited to the gateway, which had doors, excluding the porch which had none. The thickness of the north and south walls, after allowing for a passage equal in width to the length of the thresholds, is $2\frac{1}{2}$ cubits.

The length of the gate inclusive of porch (verse 15) was 50 cubits; the east and west walls of the guard-room portion will be found, after deducting the size of the porch and the other dimensions already given, to be 6 cubits thick.

In the fourteenth verse is mentioned the measurement that fixes the projections of the gate in relation to the enclosing walls, and, as will be afterwards seen, the ultimate form and dimensions of the entire enclosure. Hitherto, Ezekiel had been describing linear measurements, but now the expression "post of the court round about the gate" may be taken to imply that the prophet's companion made a *girth* measurement from the post of the court on one side right round the gate to the post of the court on the other side, of 60 cubits. By deducting the girth of the porch, which is 45 cubits (see author's plan from A to B), from this 60 cubits, 15 remain, or $7\frac{1}{2}$ cubits for each shoulder.

Much ingenuity has been employed in interpreting the meaning of this passage. Dr. Keil propounds the very singular arrangement of lofty pillars, 60 cubits high and 2 cubits square, as flanking each side of the porches, and justifies this inference by the opinion of Kliefoth, who thought that as obelisks, minarets, and factory chimneys rest on a narrow base, so there was nothing unreasonable in applying this conjecture to the Temple.

H. B. D.'s commentary on the passage is: "Kliefoth followed by Heng, Keil, Schroder (Lange). Perrot and Chipiez and others, defend the text as it stands. The two 'posts' at the end of the porch were like church steeples—so says Kliefoth: and it was such gate pillars that suggested our church steeples. But the 'posts' in question formed no part of the sanctuary as church steeples usually do, unless, indeed, Kliefoth was thinking of the campanile or bell-tower churches, such as are to be seen at Chichester, &c. It is far more sensible to amend the text with the aid of the LXX and to read 'and he measured the porch 20 cubits,' that is, in breadth."

But the acceptance of the figure 20 does not settle the point, as the porch cannot by any method of calculation consistent with the text be made to turn out 20 cubits broad. Verse 9 says: "then measured he the porch of the gate 8 cubits and the posts thereof 2 cubits," or 10 cubits altogether;

this is repeated in verse 11 where the other or breadth measurement of the porch is given as 13 cubits. The porch was therefore 13 by 10 cubits according to the Revised Version.

The guard-rooms were lit by trellised windows without any glass (many exquisite examples of such are to be seen in the East at the present day), and there were also similar windows in the porch, the term "arches" in verse 16 being translated by Professor Davidson "to the porch thereof."

Ezekiel is now led from the porch to the outer court, when, looking to the right and left, he saw along the inside of the east wall thirty chambers, or, in other words, a colonnade divided into thirty recesses abutting on the shoulders of the porch. Though not mentioned, it may be supposed there were similar colonnades against the north and south walls.

Professor Davidson, in his *Cambridge Bible*, infers from chapter xlii. verse 6, presumably from the expression "they had not pillars as the pillars of the courts," that these chambers were of several stories; but it is not likely there were storied buildings at the outer wall, intercepting the view of the Temple from the outside; the expression more probably refers to the pillars of the chambers surrounding the Temple.

The next step was to measure the "space from the forefront of the lower gate to the forefront of the inner court without."—this was 100 cubits (verse 19)—and it is followed in verse 23 by an apparently irreconcilable measurement of 100 cubits from gate to gate. If we take the nineteenth verse to mean that the forefront of the outer gate is its eastern front, so making that gate project entirely within the wall, and measure from it 100 cubits to the east side or forefront of the inner court, and suppose that the 100 cubits in the twenty-third verse is not a *vis-à-vis* measurement, but one from the eastern face of the one gate to the eastern face of the other, we comply with both conditions, with this result that all the three inner gates meet each other, and leave no room for the altar in the central court.

Benzinger's plan, *H. B. D.*, meets the difficulty by forming an intermediate court, which, receiving all these inwardly projecting gates, leaves a clear central court of 100 cubits; but this plan has the objection that the eastern exit from the priests' chambers, described in chapter xlii. 9, would not open into the *outer* court according to the requirement of the passage. Nor again would the little altars be in the *outer* court as described in chapter xl. verses 40, 41, but in the *intermediate* court. Besides, if the inner court of the altar was the priests', to what use was this extra court put? for at that time there were none of the after-distinctions of Priests, Israelites, and Gentiles, for whom separate courts had to be provided; while in any case Ezekiel makes mention of only outer and inner courts. And, moreover, the surveillance exercised by the inner guard-rooms over this intermediate court, with its area of 26,000 cubits, would be disproportionate to the control of the three outer guard-rooms over the outer court which had an area of about 200,000 cubits.

We assume, therefore, as the only arrangement meeting all the conditions of the text, an outer court 100 cubits wide, and a distance of 100 cubits from the exterior of one porch to the exterior of that opposite.

As the outer porch, according to our interpretation of the fourteenth verse, projects $7\frac{1}{2}$ cubits into the outer court, this makes the inner porch project $2\frac{1}{2}$ cubits into the inner court. The latter dimension provides for the architectural necessity of separating the porch corners from the surface of the adjoining wall.

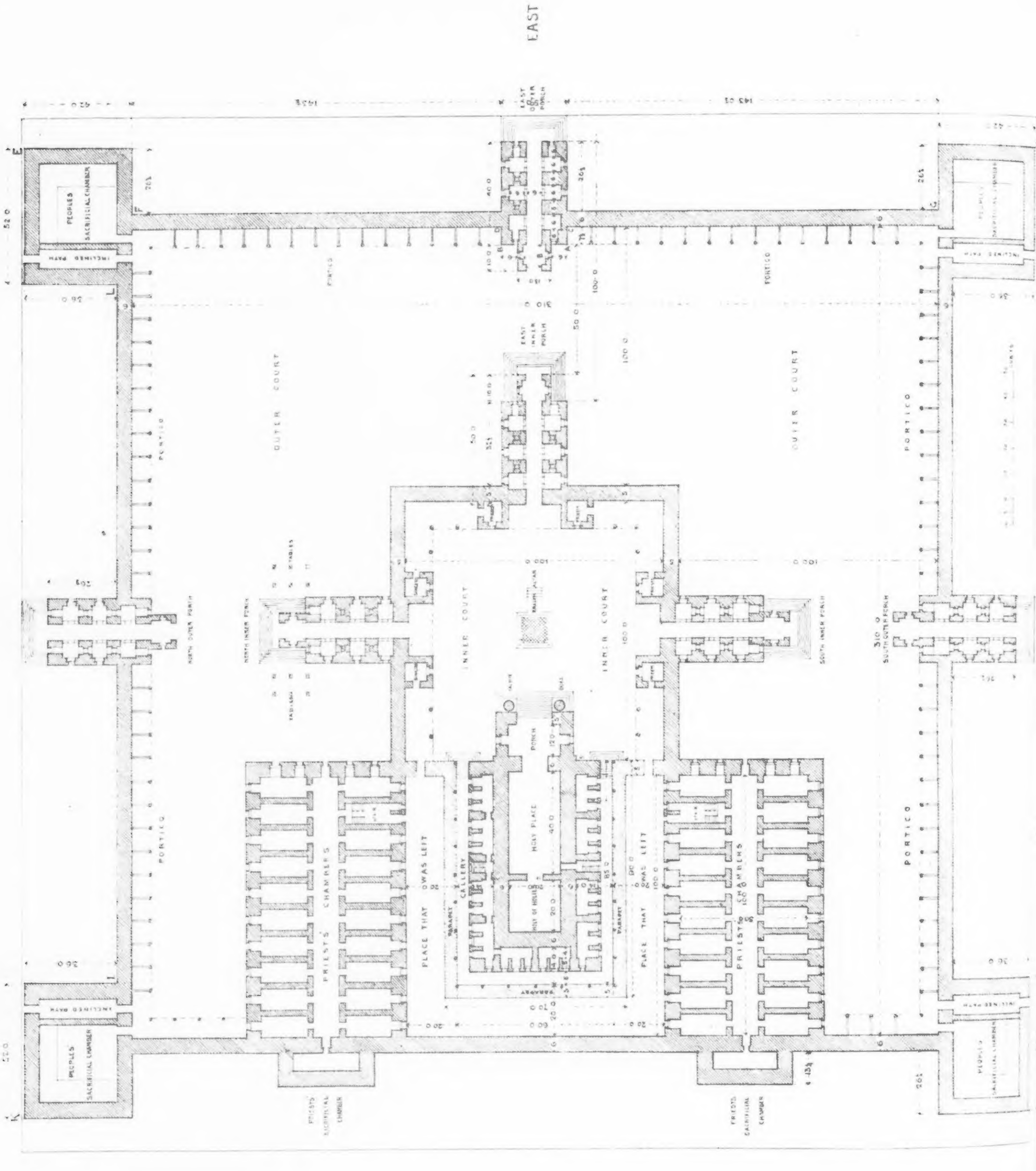
The theory of the outwardly projecting gates is confirmed by the reference in verse 18 to "the pavement by the side [or shoulder] of the gates," "even the lower pavement." In the *H. B. D.* plan this lower pavement has been shown as inside, and next to the enclosing wall, with the obvious disadvantage of serving as a canal in the wet season. In the author's plan, such a pavement "by the side of the gates over against the length of the gates" would not have such a defect, and might very properly be designated a lower pavement, because it was lower than that of the outer court. The Ezekiel Temple was not an isolated building in the open country with its containing walls rising at once from the natural surface of the ground, but, according to the later chapters of the prophecy, was in the centre of a complexus of enclosures, and therefore requiring such a pavement as a means of adaptation to its surroundings.

Ezekiel and his companion next examine the north and south external gates, which agreed in all respects with the eastern gate.

They now measure the south, east, and north inner gates, and these are seen to be similar in form and dimensions to the outer gates, but their porches were turned outwards to face the porches of the outer gates, according to the passages in verses 31 and 34 "and the arches (porch) thereof were towards the outer court," and also in accordance with the passage in chapter xlvi. 2, where we read that the Prince on the Sabbaths and New Moons was to "enter by the way of the porch of that gate without, and shall stand by the post of the gate," "and he shall worship at the threshold of the gate."

All the inner gates were ascended to by eight steps, the outer gates by seven, and together these two flights likely had some symbolical relation to the fifteen Psalms of Degrees.

In the thirtieth verse is a reference to arches 25 cubits long and 5 cubits broad; this verse is omitted in the Septuagint, and Dr. Davidson thinks it "may have arisen from an inaccurate repetition of the measurements given in previous verse."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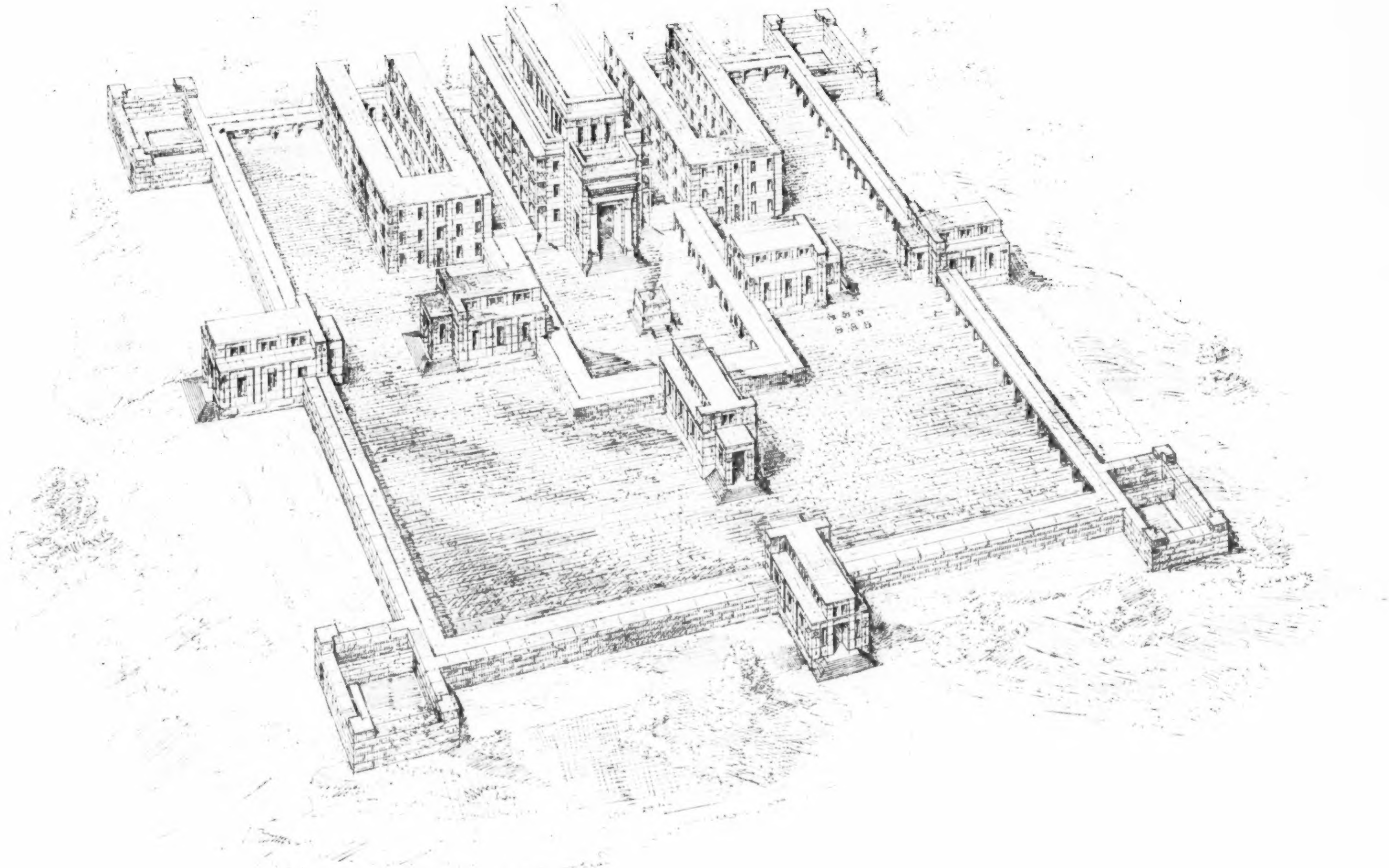

ISOMETRICAL VIEW OF THE TEMPLE OF EZEKIEL

But taking Dr. Keil's translation of the word "arches" as meaning "wall projections," and admitting for the present purpose the genuineness of the passage, we suggest that it may refer to some architectural frontage on the gate which returned with equal distinction on each side for a space of 5 cubits, and this, we submit, may have been a parapet on the front part of the gateway with a return of 5 cubits at each end; such a detail would give dignity and completeness to the gate façade.

This idea is introduced in the accompanying view, and the result, in conjunction with the clerestory, is the figure of a "Tau" cross, always used in representations of Moses holding up the brazen serpent. While it may be going too far to assert intentional symbolism in this unexpected figure, it furnishes a very interesting coincidence in its appropriateness in a Temple erected to carry on the worship of the God of Israel according to the Mosaic Ritual.

In verse 47 the size of the inner court is given as 100 cubits square, and allusion is made to the great altar which stood in the centre. The Temple was on the west side of the court, and the prophet is now led to it, and makes record of the dimensions given him.

Beginning with the porch on the east side, we are told (Ezek. xl. 48, 49) that it was 20 cubits long by 11 cubits broad (the LXX gives 12), its walls 5 cubits thick, and as the piers or posts of the entrance were internally 3 cubits broad each, this left 14 cubits for the width of the opening; on the outside of these piers were pillars, very probably those known in the earlier Temple as "Jachin" and "Boaz." The porch was lit by windows on each side (chapter xli. verse 26), and was ascended to by the steps, as the Septuagint renders verse 49, chapter xl.

In chapter xli. begins the measurement of the Temple itself. The entrance door is mentioned in the second verse as 10 cubits wide, and the jambs on each side 5 cubits each, thus making up the 20 cubits of width. The posts or thickness of the east wall are 6 cubits (verse 1), corresponding, as we note in the fifth verse, with the thickness of the remaining walls of the Temple. After measuring a length of 40 and a breadth of 20 cubits as the size of the "Holy Place" (verse 2), Ezekiel and his guide reach the veil of the "Holy of Holies"; its thickness is mentioned as 2 cubits, the door 6 cubits, and the jambs 7 cubits wide (verse 3), so between them making up the Temple width of 20 cubits.

The "Holy of Holies" is mentioned as 20 cubits square (verse 4). The measurement of 4 cubits as the width of the chambers surrounding the Temple seems very narrow, but it would be increased by the window recesses formed in the outer wall which was 5 cubits thick. The chambers were three stories in height, ten on each floor and thirty in number altogether. The ninth verse gives 5 cubits as the breadth of the wall or terrace all round in front of the chambers, and its height is alluded to in the previous verse as a full reed, or 6 cubits above the inner court.

It will be noted that nothing is said of the altitude of the Temple, or of the other buildings, the only instance in which heights are given being those of this terrace and of the main enclosing wall of the outer court.

The tenth verse gives the width of "the place that was left" between the 5-cubit ambulatory and the enclosing court wall as 20 cubits. *H. B. D.*'s plan continues the "place that was left" round the porch and omits it at the west end; but this arrangement is incorrect, as the passage reads "the breadth of the place that was left was 5 cubits round about," that is, round about the chambers, not round the porch; besides, the architectural effect of the porch standing on such a platform would be unpleasant; but this objection could not apply to the chamber platform, because by supporting columns connected with the galleries above, it had evident continuity of outline which the porch with such a base could not have.

The twelfth verse alludes to a building at the front of the separate place at the west as 70 cubits broad and 90 cubits long, with its walls 5 cubits thick. *H. B. D.* quotes an opinion as to its use in the following words: "Kliefoth and Keil hold that the 'binyan' (or enclosed space) was made for the purpose of receiving the offal of the sacrifices and the sweepings of the gates." While recording this opinion, *H. B. D.* does not apparently approve of it, and rather haltingly remarks: "It is very probable that by the 'binyan' we are to understand the same as the passage in 2 Kings xxiii. 11, as a place where horses and chariots were kept; and in 1 Chronicles xxvi. 18 (a part of the temple, west of the house of which the priests had the charge)." But we suggest that the dimensions 90, 70, and 5 cubits simply refer to the ambulatory parapet on the two sides and west end of the Temple, which on the author's plan is exactly of these sizes, and that there was therefore no such area west of the Temple at all.

The accuracy of this interpretation is borne out by the words in the next verse, "so he measured the house an hundred cubits long, and the separate place and the building with the walls thereof an hundred cubits long," a repetition of former measurements which makes no allusion to any such large area, as it would surely have done in the *résumé* had such a space existed. And besides it is very unlikely the Mosaic Ritual would permit the presence of such an extensive insanitary enclosure within the precincts of the Temple.

The 100 cubits of the thirteenth verse given as the length measurement of the Temple is the exact product of the figures marked on the author's plan, if we accept the emendation of 12 cubits given by the LXX instead of the 11 of the Revised Version (chapter xl. verse 49).

The sixteenth verse alludes to windows: these must have been the lights to the "Holy Place" rising above the roofs of the porch and chambers in clerestory form.

The twenty-fifth verse refers to thick beams of wood placed on the outside of the porch, very probably as a cornice to the two brazen columns mentioned in chapter xl. verse 49, and protected from the weather by a covering of gold, for it is hardly likely the columns stood in isolation as assumed by the late Mr. Fergusson, who wrote so ably on the Jewish Temple.

Chapter xlii. begins with the description of a group of three-story chambers which stood in the outer court opposite the Temple buildings, coinciding in length with the 100 cubits of the north side of the Temple court, and extending from that side northwards 50 cubits into the outer court. They were for the use of the priests, who went there after duty in the Temple court to change their official garments, ere they passed in among the people into the outer court; and also as a place for depositing the "meat," the "sin," and the "trespass" offerings. The priests were not allowed to go direct to the outer court (chapter xlii. verse 14), and therefore a door was provided from the inner court to the chambers (verse 2) through which the prophet went from the "inner" court, the LXX so translating the word "outer" in the first verse.

Inside he sees a walk of 10 cubits broad by 100 cubits long (the figure 1 in verse 4 should be 100 according to Professor Davidson), with an exit at the east end to the outer court (verse 9), and an entrance also at the west end to the Priests' Sacrificial Kitchens (chapter xlvi. verses 19, 20). When the priests needed to go on duty into the outer court, the eastern door would allow them to pass at once into it.

The chambers were on the east, north, and south sides of the block, so that the priests would be screened from the gaze of the worshippers in the outer court as they went to and fro their rooms. The upper rooms were approached by galleries, which, not being supported on pillars like those of the Temple, receded at every story, so that the rooms on the middle floor would be about 4 cubits narrower than those on the ground level: and the rooms on the third story correspondingly narrower than those on the middle stage. The stairs to them would be conveniently situated near the eastern door and the door into the inner court.

The chambers of the south side (verses 10, 11, 12) corresponded with those on the north side. It will be noticed on the author's plan that the ground floor of the two chamber blocks is sunk below the level of the inner court, so that the priests would have only a few steps of ascent to reach the middle tier of rooms which were little above the level of the inner court.

The measurements of the buildings within the Temple courts being completed, Ezekiel is brought through the eastern gate to the outside of the enclosing north, south, east, and west walls, which are measured in his presence and found to be 500 reeds, or, as corrected by the LXX, 500 cubits each.

To meet this statement *H. B. D.* represents the Temple area as enclosed with a straight-lined wall, which, unbroken in outline, necessarily leads to so large an internal area as to require a greater number of courts than the inspired record allows.

The author's plan, on the other hand, measuring round the broken outline which is obtained by adding the porches already described and the "Peoples' Sacrificial Kitchens," 40 by 30 cubits internally (chapter xlvi. verse 22) or (adding thickness of wall) 52 cubits by 42 cubits externally, secures the desired dimension of 500 cubits for each side, the Priests' Kitchens (chapter xlvi. verses 19, 20) being substituted on the west side for the porches of the other three sides.

Another problem, more related to the Solomonic than the Ezekiel Temple, which has excited much attention and is partially referred to in the *E. B.*, page 4935, vol. iv., concerns the columns and their capitals. According to the statement in 1 Kings vii. 15 and Jeremiah liii. 21, the shafts were 18 cubits long. In 2 Chronicles iii. 15, they are said to have been 35 cubits, and the capitals in all the other instances 5 cubits high. They were 4 cubits in diameter, as we learn from the expression in 1 Kings vii. 15, "a line of twelve cubits did compass them about." The verse in 2 Chronicles clearly refers to the united length of the two shafts without their bases; in this way, we may reconcile all the dimensions of length in the three passages cited. They were with their capitals 23 cubits high, and therefore $5\frac{3}{4}$ diameters in proportion of diameter to height, and so rather approaching the vigour of the Grecian Doric column than the grace of the Corinthian.

The capitals were bowl-like in form (1 Kings vii. 41), apparently resembling some of the later Byzantine capitals, and these bowls were covered with net or lattice work; and as the net work, being 4 cubits or so high, would present a monotonous surface, it was relieved by seven rows of wreathed chain work. The summits of the capitals had "pommels," or what we may understand as some kind of volute, provided to carry the plan outline of the capital bell from the circle to the square. From each of these pommels were suspended, after the manner of a festoon, two rows of pomegranates, one hundred in each row, or, according to the description in Jeremiah lii. 23, ninety-six towards the four winds – in other words, that number on each face, leaving four over on each festoon for suspension from the pommels. This combination of details is reasonable, and would form a capital in harmony with the sturdiness of the shafts and possibly full of symbolism.

The position of the columns in relation to the

porch has been the subject of much discussion, and, although the theories propounded have their application rather to Herod's Temple than to Solomon's, they have an interest in our present consideration to this extent, that the pillars of the late Temple were survivors of the earlier. The late Mr. Fergusson, author of several important works on Architecture, maintained that they were incorporated with a series of lintels after the form of the Sanchi Tope in India, and that on these lintels was hung the sacred vine; this piece of construction he placed outside the Temple in the same isolated position as the Japanese Torii. This idea Mr. Fergusson takes from, or finds confirmation of, in the Talmudic statement that the porch had five carved oak beams or lintels with stones in between, and that these beams increased in length from the lowest to the highest—that is to say, each beam was longer than the one below it by 2 cubits, so that the top beam would be 8 cubits longer than the bottom one. An objection to Mr. Fergusson's theory of isolation occurs in the reference to intermediate stones, which, appropriate enough in a structure placed against the Temple walls, would be out of place in one standing alone. An alternative to this is that the columns might have been supports to the porch lintel; and that this is not shut out of consideration is seen by the passage in I Kings vii. 19, from which we learn that the columns were set up *within* the porch; this being so, the use of a series of wood lintels with stones between could be understood, as they would cover the void of the porch opening, on the same principle as the five separate courses of stone placed over the chamber of the Great Pyramid relieved it of the dead pressure of the superincumbent mass. Plated with metal, these wood lintels would resemble a crown on the summit of the pillars, somewhat like a stone which surmounts the lintel of a Temple at Baal-Zimmon. This assumed application of gold would help to explain a statement of Josephus, that the front of the Temple shone from afar with the reflection of the gold which covered it.

A late writer, M. Chipiez, the learned French author of several architectural works, conjointly with M. Perrot, assumes the columns to have stood alone, and considers this assumption strengthened by the collateral evidence furnished through the discovery of a fragment of sepulchral glass, which was found in 1882 on the Labican way near Rome, and which it is supposed was due to a Hebrew colony in Rome that flourished there in the end of the third and the beginning of the fourth century A.D. On this fragment is represented a view of a temple with isolated columns standing at each side of it. The temple is localised by the representation of palm-trees, and by a view of the seven-branched candlestick below it. This picture is supposed to be a Jewish reminiscence of the ancient Temple of their fathers, and, if that be so, it would go far to support his theory that the columns stood alone and in advance of the porch, in a position similar to the obelisks which were so invariable an accompaniment of the Egyptian pylon. The French author goes on to give most elaborate drawings of one of the columns, of which all that need be said is that they are French in treatment, and remind one of the practice of some of the early painters, who, in their ignorance of Eastern costume and scenery, represented sacred objects amidst the accessories of their own people and country. On the whole, it will be preferable to suppose that the columns were on the outside, immediately against the wall.

There is a matter of incongruity in *H. B. D.* (page 702) commentary on the passage in I Kings vi. 36, and with an effort to confute this our remarks must close. The text reads thus: "And he built the inner court with three rows of hewn stone and a row of cedar beams." *H. B. D.* explains that this means a wall of three courses of stone, finished with a coping of cedar, weathered to throw off the water. But why should a stone wall have a coping of a material so much less durable than itself? Would the upper course of stone not have been better as a coping? And, besides, to provide a wood coping two or more cedar planks would have been needed. The breadth of the wall was probably 5 cubits, or 7 feet 6 inches, and the largest cedar in Solomon's days would likely be no bigger than the trees Dr. Thomson describes in his *Land and the Book* as attaining the circumference of 41 feet, dividing into two or three stems at the height of a few feet from the ground. How could these planks be joined together so as to be watertight? Dr. Keil thinks they stood upright like a railing, but this seems an undignified method of fencing when metal was available.

The proper resolution of the difficulty is, we believe, to be found by interpreting the passage as meaning that the north, south, and west sides of the Temple chambers had each a row of hewn stones (three rows of hewed stones) as a pathway projecting from the chambers, and that on these rows rested the wood pillars mistakenly supposed to have been copings which supported the gallery beams, these in their turn carrying the passages of approach to the chambers.

It may be added that it is very unlikely that the rooms were entered by passing from one into another as *H. B. D.* plan shows; this would entirely destroy their privacy as places of abode for the priests.

REVIEWS.

ENGLISH MEDIÆVAL SCULPTURE.

An Account of Mediæval Figure Sculpture in England. With 855 photographs. By Edward S. Prior, M.A., F.S.A. [F.], Slade Professor of Fine Art in the University of Cambridge, and Arthur Gardner, M.A., F.S.A. 4o. Cantab. 1912. £3 3s. net. [Cambridge University Press, Fetter Lane, E.C.]

No one can take up a work which has on its title-page the name of Professor Prior without an expectation that all available data of the subject selected will have received the most thorough investigation, nor without confidence that every conclusion arrived at will be based on the most profound and reliable consideration of the premisses. In the present case Professor Prior and Mr. Gardner have more than fulfilled every possible expectation. The book which they have given us is not only the first attempt at a systematised history of English Mediæval sculpture, but it will doubtless remain for many years the principal authority on this extremely complicated and interesting subject.

The task which M. Emile Mâle undertook when he produced, ten years ago, his work on the religious art of the thirteenth century in France was, we venture to think, a far easier one. It is, indeed, true that the Gallic lucidity of M. Mâle's method makes it difficult to realise the laborious preparation which must have formed the foundation of the logical system which he sets before us. But it is also true that the body of French sculpture which is available for study is at once sufficient as regards its quantity, and complete enough as regards its logical arrangement and development, to make its interpretation comparatively simple when once the master key of its symbolic intention has been discovered.

Very different is the case of English sculpture; even before the iconoclasm of the sixteenth and seventeenth centuries had done its work we doubt if it would have been an easy task to interpret and systematise as a whole the symbolic intention of English Mediæval sculpture. The confused mass of disconnected fragments that now remain are in themselves almost beyond classification: it is a matter of wonder that any one should have had the courage to attempt it, and of still greater wonder that it should have proved possible to put together so remarkable a co-ordination of the results of an almost infinite capacity of accurate observation as is presented by the volume before us.

The ordered mass of sculpture on the façades of the great churches of Amiens, Rheims, Paris, and Chartres is as complete, or nearly so, as it was in the thirteenth century. The general lines of interpretation were indicated by Mr. Ruskin some forty years ago. The system of interpretation was carried further, and, indeed, reduced to logical completion, by M. Mâle; it was based upon a study of the sermons and other literature of the thirteenth century, a study of the books which were read and of the sermons which were doubtless actually heard by the men who carved the figures and bas-reliefs which are clearly shown by M. Mâle to be their direct interpretation.

The only churches in England comparable to the French churches in the quantity and quality of their sculpture are those of Exeter and Wells. The Wells sculpture is thought by the authors to be as early as the corresponding French work, if not earlier, but in both cases the preservation is far less complete, and in neither case was the symbolism realised to the same extent as in France. It is hardly too much to say that, without French sculpture as a guide, the interpretation of English sculpture would have been almost impossible: even with its help the result must be comparatively uncertain and fragmentary.

An actual instance may make this clear. The imagery of the west front at Amiens is arranged as follows:—The central porch contains the figure of Christ, surrounded by the twelve Apostles; the south porch, the Blessed Virgin with S. Elizabeth, S. Simeon and S. Gabriel, and other figures illustrating her history. The north door is devoted to a group of local saints and martyrs; the front of the buttresses between the porches, to a series of prophets who foretold Christ. Above, in the tympanum of the three porches, the life of S. Firmin, the Last Judgment, the death and burial of the Blessed Virgin; in the voussoirs the hosts of heaven; and above again, in the arcade immediately under the towers, the kings of Judah, the ancestors of Christ; and lastly, underneath, on the plinth, the Zodiac and the corresponding labours of the seasons; the virtues and vices, and a number of historical scenes and incidents—a complete and logically ordered picture of Mediæval thought, which can be classified, as M. Mâle classifies them for us, under the four heads of the Mirror of Nature, the Mirror of Science, the Mirror of Morals, and the Mirror of History.

A very similar and not less complete arrangement of sculpture is to be found at Rheims, at Paris (restored and largely modern), and at Chartres. In each case the arrangement is so clear and logical that it is possible without much difficulty to identify every figure. This is seldom possible in England. Only fragments of such an ordered scheme can be identified here and there. Messrs. Prior and Gardner are able, indeed, to identify at Peterborough and Wells and Exeter, Apostles, kings and ladies, the Messengers and ancestry of Christ. At Wells, Professor Lethaby has been able to discover a scheme of imagery approaching in its grandeur and completeness those of the great French churches; while here and there all over the country are to be found isolated examples of zodiacs, judgments or dooms, resurrections and coronations, and various historical episodes.

In the present work, however, a point of view

more fully treated than that of the interpretation of the subject-matter is that of craftsmanship, and the originals and classification of the various schools and methods of imagers and masons. This most difficult subject is treated with extreme thoroughness, and it is difficult to realise how any two men can have found time to acquire such intimate knowledge of so large a number of examples of sculpture as to be able to present an orderly classification of what are, at first sight, isolated and disconnected fragments.

Nothing in the whole work is more interesting than the disentangling of the various sources to which the varying characteristics of, for instance, the early and late Romanesque sculpture in England are to be attributed. Continental influence in the South, and Irish and Scandinavian influence in the North and Midlands in Saxon times, and, later on, a more clearly indicated French influence which came through Poitou and perhaps through Santiago de Compostella, are followed out with considerable detail: while the classification of the several schools of masonry and sculpture which came into existence in the twelfth and thirteenth centuries is of the greatest interest.

The book is very amply illustrated with more than eight hundred photographic prints.

ARTHUR S. DIXON [*F.*].

CORRESPONDENCE.

Book-names for Building-work.

15th April 1913.

To the Editor, JOURNAL R.I.B.A.,—

DEAR SIR,—My friend Mr. Walter Millard in the JOURNAL, 8th March, p. 311, voices the opinion of many students of "Gothic" (may I use the word Gothic ?) as to the desirability of giving up sectional names like "Early English," "Decorated," and "Perpendicular." I cannot prove them to be wrong if they wish to do so, but I think it would be a pity. I remember that a wish to "discriminate the periods" gave an interest to my early reading, and I don't think I should have attached the general characteristics of an ever-changing process so readily (it was difficult at best) to mere dates. Surely it would be generally allowed that in all "arts and sciences" (generally so called) names help. There are no bears and swans in the "heavens," but it is probable that the retention of the old names for the "constellations" (there are no constellations of course in fact) has a practical value. It is possible that our first day of "summer" may be colder than the last day of "winter," but, notwithstanding, it is convenient to have distinct names for the continuously changing "seasons" (so called). Where are we to end in giving up convenient labels ?

Another friend of mine, Prof. Moore, thinks that we "English" (are we English ?) are wrong in applying the word Gothic to our "Mediæval" (when did the Mediæval period begin and end ?) "architecture": must we agree ? One argument for giving up the sectional labels is brought forward; this is that the French get on very well without such nicknames. It is perfectly true, but a moment's reflection suggests that the modern nation of France was not in the middle ages a single homogeneous kingdom. The same "styles" (what are styles ?) did not run from Provence to Flanders and from Savoy to Brittany. In England the styles marched very well together from corner to corner. Besides, the French may do as they like. It is my view that after a hundred years of printing stuff about English architecture a general notion is getting about that twelfth-century architecture was "Norman," that what is called "Early English" was developed in the thirteenth, the "Decorated" came in the fourteenth, and the "Perpendicular" followed in the fifteenth century. I believe it would not be wise now to repudiate the results of endless writing and infinite speech. For myself I confess that I quite spontaneously think of such a porch as "Norman" and such a window as "Perpendicular." For others there is no right or wrong in the matter: it is only a question of convenience.

I foresee that in a few years we shall only be able to write on "architecture" at all by putting every third word in quotes, as no one will accept any of the names in current use.

W. R. LETHABY [*F.*].

Colour Decoration.

14th April 1913.

To the Editor, JOURNAL R.I.B.A.,—

SIR,—I suppose that an author should always take any criticism "lying down"; but where his meaning is distorted by words inserted by the critic, he is apt to kick.

I have to thank Mr. Ibberson for many appreciative remarks on my book on *Colour Decoration*, but why does he ask "can one altogether agree with this, of the much abused ***easel*** picture ?" and then quote a passage in which I am speaking exclusively of *mural paintings*, painted with the object of beautifying a building—not of pictures *hung up* in it ? The word "***easel***" is his own insertion. I am not wanting in admiration for easel pictures, but I am not talking of them. I am discussing decoration. His three Vandyke portraits may be sold at Christie's and go perhaps to America —I am discussing what are loosely called "frescoes"; but as that word indicates a ***method***, I did not use it.

As to imitations, I am all for the real thing; but so long as architects build sham columns of plaster to conceal iron stanchions, or to obtain "dignified effect," why try to foist all the lie on to the

decorator who tries to make the best of the original untruth? Who is most to blame—the architect who built cement pilasters as responds to granite columns, or the decorator who saw what the architect meant, and did it?

And in a humbler matter, where is the architect who never specifies for the servants' rooms, "woodwork, three coats *stone colour*"?

Alas, it is a wicked world!

J. D. CRACE [*Hon. A.*].

Mr. Ibberson, to whom a proof of the above was sent, replies :—

21*st April* 1913.

In case I have failed to make my meaning clear *to others*, may I reply in the JOURNAL to Mr. Crace's gently given kick?

I certainly dragged in "easel" wrongly and unnecessarily; the word is not needed for my point that three balanced Vandyke portraits, "painted on the wall," like Browning's duchess, might be "decorative" even though surrounded by a definite gold frame or border. Possibly Mr. Crace would admit that the effect was "beautiful" and we are but kicking against the words?

In vicarious humbleness I admit that the architect of the "Fitzbilly" sinned: a granite column really should not have had a plaster respond! (It was putting too great a temptation in the way of the painter).

But even architects, poor slaves of material things, must not lie down always, and I proceed to adopt the erect posture of dogmatism. Plaster pilasters are legitimate when the columns are plaster too; the simplest of us knows they are not plaster all through, and whether there is *stone* behind as sometimes at Pompeii, or *brick* as in Georgian times, or *reinforced concrete* as now, there is no real deception. The columns after Mr. Crace had worked his wicked will would I expect deceive the angels.

In conclusion, I must again resume a recumbent position and plead guilty to using "stone colour" on my kitchen doors. I do not think the cook is deceived, but should I at any time have the pleasure of building for Mr. Crace, I will remember the risk of this apparently innocent habit deluding more sophisticated minds, and will specify instead, in the words of another decorator, "All the colours of the rainbow including black and gold."

HERBERT G. IBBERSON [*F.*].

Books Received.

London County Council—Survey of London. The Parish of Chelsea (Part II.), being the fourth volume of the Survey of London, by Walter H. Godfrey, member of the Committee for the Survey of the Memorials of Greater London. 4o. Lond. 1913. Price 15*s.* 9*d.* Published by the London County Council, Spring Gardens.

The *Country Life* Book of Cottages, costing from £150 to £600. By Lawrence Weaver. 8o. Lond. 1913. Price 5*s.* net. [*Country Life* Office, 20 Tavistock Street, Covent Garden, W.C.]

9 CONDUIT STREET, LONDON, W., 26*th April* 1913.

CHRONICLE.

The late William Flockhart [*F.*].

The formal announcement to the Institute of the death of Mr. William Flockhart was made by Mr. E. GUY DAWBER, acting for the Hon. Secretary, at the General Meeting of the 21st inst., in the following terms: I deeply regret to announce the decease on the 10th inst. of our distinguished Fellow and Member of Council, William Flockhart. Mr. Flockhart was elected a Fellow in 1901, and had always taken an active share in the work of the Institute. He had been seven times elected Member of the Council, and in that capacity had served for three years on the Finance Committee, acting for a time as its Chairman. He was also for several years a member, and for the past two years Vice-Chairman, of the Art Standing Committee. Mr. Flockhart's artistic gifts are well-known to members. He has left his mark especially on our West End architecture in a number of charming buildings, business premises and private houses, in Bond Street, Hill Street, and elsewhere in this neighbourhood. As architect to the Union Castle Steamship Company he made the designs for the remodelling, furnishing, and decoration of the Royal quarters on the *Balmoral Castle* for the voyage of the Duke of Connaught and suite in 1910. He was one of the architects selected by the London County Council to submit designs for the façades of buildings fronting the Strand and Aldwych, and he received the second premium awarded in that competition. He was also among the nine architects nominated to take part in the final competition for the Wesleyan Church House, and one of the eight selected to submit designs in the final competition for the new London County Hall. I need not on this occasion deal more fully with his professional works—these will doubtless be set out at length in the Memoir to appear in our JOURNAL. We have to-night to deplore the loss of a good and sincere friend of the Institute. Mr. Flockhart was an able and brilliant architect; he was, moreover, a competent critic, tolerant and broad-minded, and a passionate lover of the arts. He had a most lovable disposition, and was of a peculiarly winning personality. Those of us who enjoyed his

acquaintanceship know how charming a companion he was, and we all regarded him as a friend. I have to move the following resolution: "That the Institute desires to record its deep regret at the decease of our esteemed and distinguished Fellow and Member of Council, William Flockhart, and at the loss which the Institute and Architecture have sustained thereby; and that a message of condolence be transmitted on behalf of the Institute to his widow and family, sympathising with them in their great bereavement."

Mr. GEORGE HUBBARD, F.S.A. [*F.*]: May I be permitted to second this vote of condolence on the loss of our friend, Mr. Flockhart? I think he must have been known to all of us in this room, and all who knew him must have respected him, not only as an architect, but as a man. I had to meet him professionally on more than one occasion, sometimes as an opponent, and it would have been impossible to have dealings with a man of a more generous or a sweeter disposition. While loyally supporting his own case, he would be most considerate of the views held by others. It is with extreme regret that we have heard of his death, and we shall always mourn his loss.

The PRESIDENT: In putting this vote of condolence I should like to say how deeply and sincerely we all sympathise with Mr. Flockhart's family. Mr. Flockhart was one of the most delightful of men, and possessed of great charm and originality, as well as a very individual outlook on things. He was also an artist of considerable power, and a brilliant draughtsman. On his services to this Institute I need not dwell; we all know he was most constant in his attendance and in his efforts to help us in every way. He was a loyal colleague, a man with a very candid and open mind, always anxious to get at the facts, and always sincere in his efforts to do his best in the interests of architecture.

The motion having been put from the Chair, the Meeting rose and signified its assent upstanding.

Collaboration of Architect, Sculptor, and Painter.

During the year just past the Committee on Allied Arts of the American Institute of Architects have been engaged investigating conditions existing among architects, sculptors, and painters in connection with their collaborative work. The Committee was organised on democratic lines, each art being represented by a prominent member of its own cult. The result as regards the composition of the Committee is stated by the Chairman to have been highly satisfactory, and he suggests the addition to its personnel of a representative of the landscapist's art. The report recently published opens with an expression of the Committee's regret that it found in recent American architecture, particularly in the *ensemble*, so little evidence of the successful collaborative effort of architect, sculptor, and painter that it hesitated to proceed on the basis that their arts were, in fact, allied in anything but name. They recognise the seriousness of this condition, both in the loss to the arts in question of their rightful share in the architectural work of the country, and of the loss to the country itself of its birthright, a finished architecture. The Committee attribute the trouble to lack of education—that is to say, special education in sympathetic collaboration. At present such education seems confined to individual experiences, and in the work of most architects experiences involving collaboration with sculptor and painter are unfortunately extremely rare. It is not enough that the sculptor, painter, and architect should realise the necessity for unselfish collaboration—they must be taught how it may be had. The Committee feel that in this matter of education lies both the cause and cure of the trouble, and recommend that the attention of the American Institute's Committee on Education be directed especially to this lack of co-operative study. They suggest that the Institute should foster to the utmost the sympathetic co-education of the Allied Arts throughout the country, and that it should take steps to get the Trustees of the Academy at Rome to give special encouragement to such collaborative education in that institution. They further advocate the establishment by the Institute of an annual money prize for the best accomplishment in third-year collaborative work at the Academy in Rome. The report concludes with the hope that opportunity for such collaborative education may be early provided, and that through its agency there may come into American architecture that something which it now lacks and which is only found where sculptor, painter, and architect have learned to merge their several individualities in a common love for a great ideal.

London University: a Proposed University Quarter in Bloomsbury.

The Final Report of the Royal Commission on University Education in London (appointed in May 1910) is now issued as a Blue Book [Cd. 6717]. Briefly, the task set the Commissioners was to examine the existing provision for University education in London in the light of what they thought ought to exist, and to make practical recommendations towards the realisation of the ideal. The Commissioners have come to the conclusion that the organisation of the University is fundamentally defective, and as at present constituted is not calculated to promote the highest interests of University education in London; nor do they think it capable of developing on the present lines into a University such as London ought to have. Much of the defective organisation they attribute to confusion of thought about what is essential and non-essential in University education. Discussing the essentials, the Commissioners urge: First, that students should work in constant association with their fellow-students of their own and other Facul-

ties, and in close contact with their teachers. Secondly, that University work should differ in its nature and aim from that of a secondary school, or a technical or a purely professional school. Thirdly, there should be close association of undergraduate and post-graduate work; a hard and fast line between the two is disadvantageous to the undergraduate and diminishes the number who go on to advanced work. The most distinguished teachers must take their part in undergraduate teaching and their spirit should dominate it all. The main advantage to the student is the personal influence of men of original mind. The main advantage to the teachers is that they select their students for advanced work from a wider range, train them in their own methods, and are stimulated by association with them. Free intercourse with advanced students is inspiring and encouraging to undergraduates.

The Commissioners consider that a condition of satisfactory University work is that the teaching of the University in its several faculties should be concentrated as far as possible in one place. The constituent Colleges and University Departments should be brought together in one part of London, and grouped round the central buildings of the University when they are not actually within its walls. Admitting that London, as a whole, cannot be made a University town like Oxford or Cambridge, where the University dominates the town and may consist of many separate colleges without losing its unity and identity, the Commissioners think it is quite possible to create a University Quarter in London, in which the University life and interests would grow and develop, and students and teachers alike would find themselves in the atmosphere of a great seat of learning. The creation of a University Quarter would lead to economy in administration, to increased co-operation between different departments of study in the interests of new lines of work, and to greater intercourse between students and between teachers.

As to the locality of the proposed University Quarter, the Commissioners are of opinion that the most suitable and convenient site would be found in Bloomsbury. If King's College, the new University Department of Household and Social Science, the Brown Animal Sanatory Institution, and the central University buildings were all moved to the Bloomsbury district, where they would be close to University College, the School of Economics, and the new Constituent College in Arts and Science for evening students, it ought to be possible to create in time a University Quarter which would perhaps do more than anything else to impress the imagination of the great London public and to convince them that the University was a reality.

The Commissioners recommend that the central University buildings, all to be placed in Bloomsbury, should include:—

(*a*) A great hall for University ceremonies and large educational gatherings.

(*b*) Accommodation for the Senate, for Committees, for the Principal, and for the headquarters staff.

(*c*) Accommodation for the meetings of Convocation and for its officers.

(*d*) A club-house for the Union Societies, headquarters for the Officers' Training Corps, and rooms for professors, graduates, and students.

(*e*) A central University Library, supplementing the libraries in the Constituent Colleges, University Departments and Schools.

The Commissioners express the view that it would be extravagant to provide Examination Halls for the University in the centre of London where land is expensive. Students in the Constituent Colleges and University Departments would be examined in their own Colleges and Departments. The same practice could be followed in the case of students in Schools of the University; where possible, it is important that students should be examined in familiar surroundings, and by means of apparatus to the use of which they are accustomed. For private students, if room cannot be found for them in the schools and buildings of the University, accommodation should be had by hiring, or by building, suitable halls in some accessible part of London where the price of land is relatively low. The Commissioners see no urgency for the separate provision of scientific laboratories for advanced work in the central University buildings, though they hope that the site would be sufficiently extensive to allow of their erection should they in course of time be proved necessary.

As regards the area of the University, the Commissioners think that the Administrative County is the very largest which would allow of the effective organisation desired. It is thought that the University of Berlin, with nearly 9,000 matriculated students, is already too large, and it is doubtful whether the University of London would ever be able to provide for a much larger number than this an education comparable to that of Berlin. When this point has been reached the need will have arisen for another University, and if the University of London can prepare the way for a new university in the south-east of England by encouraging the development on the right lines of educational institutions beyond its own immediate area, it will have performed a greater service to education and to the State than by attempting a gigantic organisation which would be likely to end in the arid formalism of the Napoleonic Université de France.

It is recommended that the University should encourage the erection of hostels for as many of its students as possible, the hostels to be supported by special funds, and to be under the general supervision of the University. The hostels should mostly be placed in the suburbs, where fresh air

and playing fields are to be had, and they should be so arranged as to attract students and junior teachers from different faculties and from different institutions. Accommodation should be provided in the central University buildings for the Students' Representative Council and other University societies, and headquarters for the Officers' Training Corps.

London Suburbs: New Roads and New Ideals.

A note from the London Society says :—

It must be common knowledge to most that the main roads in and out of London are totally inadequate for present requirements, and will become more and more so as time goes on, but it may not be so generally known that unless something is done, and done at once, the opportunity will have gone for ever. London is being encircled by a series of town-planning schemes, through most of which new roads will have to pass, and if these are once authorised apart from any scheme for main roads, a good road scheme will become practically impossible. A fine scheme for main roads has been drawn up by the Traffic Branch of the Board of Trade, but they have no funds or power to execute it; the Road Board have funds but no power. What is wanted is that power should be vested in some central authority to lay down such a scheme of roads and secure its adoption by all local authorities before it is too late. The Royal Academy, the Royal Institute of British Architects, the Institute of Civil Engineers, the Surveyors' Institution, and the Municipal Engineers have jointly asked to be allowed to represent their views to the Prime Minister; attention has also lately been called to the matter in Parliament by Colonel Yate and by Mr. Joynson Hicks; it is very urgent, but so far nothing has been done. A map is appended, reduced from that published in the annual report of 1912 of the London Traffic Branch of the Board of Trade, which shows clearly how these town-planning schemes will block the way of the necessary new roads unless the lines of the roads are authoritatively laid down at once.

Mr. W. R. Davidge [*A.*] writing in the current *Garden Cities and Town-Planning Magazine* observes :—

There is room for not one but many Garden Cities, and while they are growing up we must not neglect the mightier cities which, do what we will, seem destined to still further increase. If we cannot at once build cities anew, let us import the Garden City ideal into our existing towns. Let us, if we can, surround them with a belt of agricultural land or open country, and surround each separate suburb too with open spaces for fresh air and recreation. We may not be able in all cases to apply the ideal to the areas already covered by bricks and mortar, but the first essential is a comprehensive plan to ensure that the future suburbs of London—many of them still quite little villages away out in Middlesex, in Surrey, and even further still—some of them already in the grip of one of the many tentacles of the mighty city, shall be saved from the same mistakes. Let us treat the hills and the streams and the open country as a sacred trust to be preserved for the health and well-being of the community, and this cannot be done without a plan of some sort—a comprehensive plan for the future that shall guard not only our cities, but all the country round.

The Henry Jarvis Travelling Studentship.

The third paragraph relating to this Studentship on page 17 of the Prizes and Studentships pamphlet requires amendment by the insertion of the words italicised below, so as to read :—

"The candidate placed highest in the Final Competition will be awarded the Jarvis Studentship, unless *being also qualified for the Scholarship offered by the Royal Commissioners for the Exhibition of* 1851 he elects to take the latter Scholarship, in which event the Jarvis Studentship will be awarded to the candidate placed next on the list. The Scholarship and the Studentship will not in any case be awarded to the same candidate."

MINUTES. XII.

At the Twelfth General Meeting (Ordinary) of the Session 1912–13, held Monday, 21st April, 1913, at 8 p.m.—Present: Mr. Reginald Blomfield, A.R.A., *President*, in the Chair; 22 Fellows (including 10 members of the Council), 30 Associates (including 2 members of the Council), 12 Licentiates, 2 Hon. Associates, and several visitors—the Minutes of the Meeting held 7th April having been already published were taken as read and signed as correct.

Mr. E. Guy Dawber, *Vice-President*, acting for the Hon. Secretary, having announced the decease of William Flockhart, Fellow and Member of Council, reference was made to his services to the Institute, and a tribute of respect paid to his personal qualities and of admiration for his work as an architect, whereupon, on the motion of Mr. Dawber, seconded by Mr. George Hubbard, F.S.A., *Vice-President*, it was

RESOLVED, that the Institute desires to record its deep regret at the death of its esteemed and distinguished Fellow and Member of Council, William Flockhart, and at the loss the Institute and Architecture have sustained thereby, and that a message of condolence be transmitted on behalf of the Institute to his widow and family, sympathising with them in their great bereavement.

The decease was also announced of Francis George Ashwell, *Licentiate*.

The following Licentiates attending for the first time since their election were formally admitted by the President:—Henry George Baker, Henry Charles William Blyth, Alexander Clark Meston, William Herbert Rogers, Granville Edward Stewart Streatfeild, Percy John Waldram.

Papers on MODERN STEEL BUILDING CONSTRUCTION having been read by Mr. FRANK N. JACKSON [*Hon.A.*] Assoc. M.Inst.C.E., and Mr. Bernard Dicksee [*F.*], a vote of thanks was passed to them by acclamation, and discussion on the Papers was adjourned till Monday, 28th April, at 8 p.m.

The proceedings then closed, and the Meeting separated at 10 p.m.

Mr. John William Stevens [*A.*].—It has to be stated that the "J. W. Stevens" whose death was recently announced is not the Associate of the Institute, Mr. John William Stevens, of 21, New Bridge Street, E.C., but an architect of the same name and initials, a member of the York and Yorkshire Society of Architects. Mr. John William Stevens, we are happy to state, is in the best of health, and in the active practice of his profession at the above address.